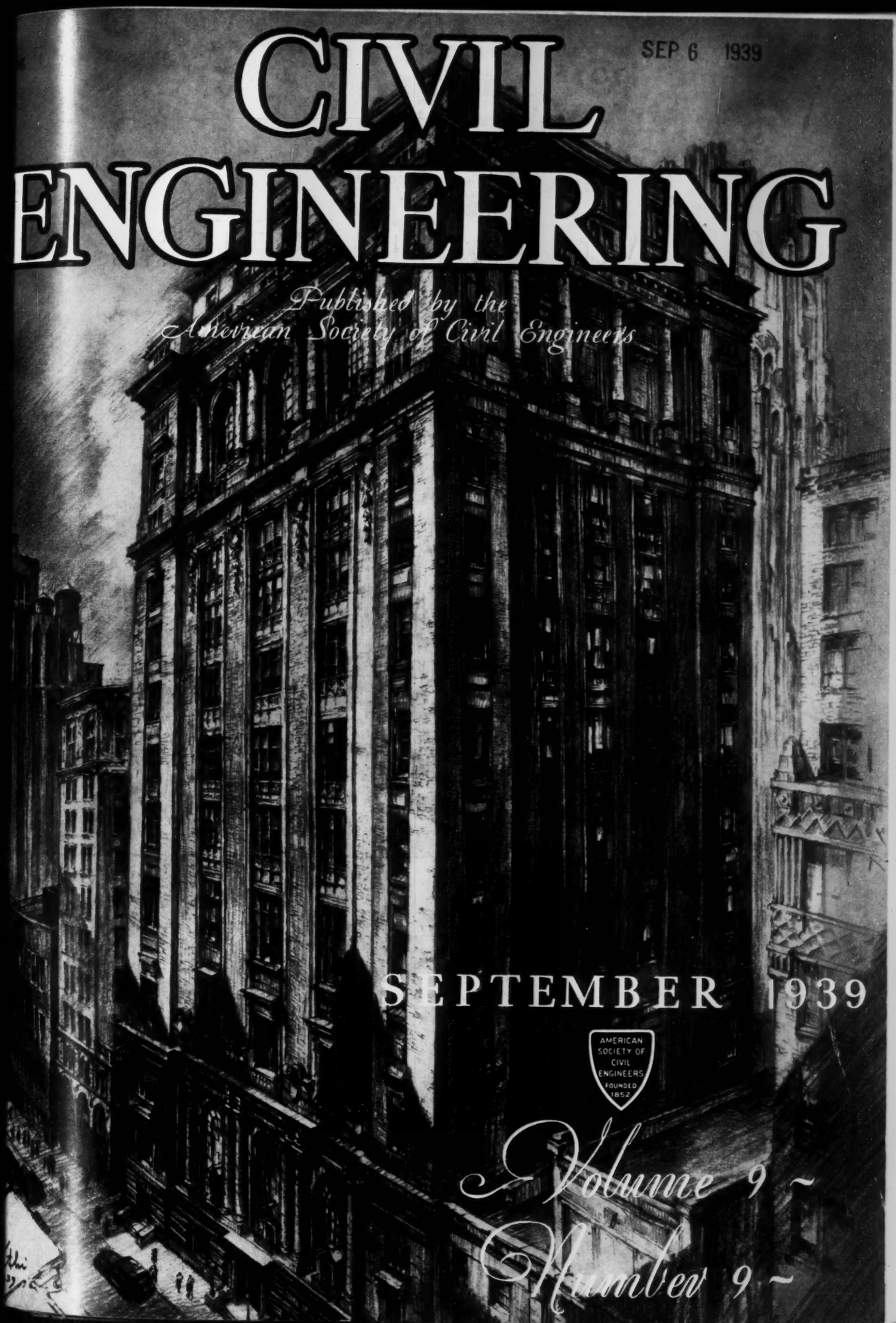
SEP 6 1939

CIVIL ENGINEERING

Published by the
American Society of Civil Engineers



SEPTEMBER 1939

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

Volume 9 ~ Number 9 ~

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HAWLEY S. SIMPSON entered engineering work with the Michigan State Highway Department and since 1925 has been engaged exclusively in traffic and transportation engineering. He is vice-president of the Institute of Traffic Engineers and this year's nominee for president. He is actively identified with numerous organizations in the field of highway traffic and safety.

ADOLPH J. ACKERMAN (University of Wisconsin, 1926) has had broad design and construction experience, largely on dams and power plants, with Stone and Webster, Aluminum Company of America, at Madden Dam (in charge of plant erection and contractor's engineering), and with TVA (as construction plant engineer). His present connection dates from 1937.

E. LAWRENCE CHANDLER, after some years in municipal engineering work, following graduation from Brown University, became connected with the Miami Conservancy District. Then followed 12 years of construction work (principally on hydro developments) and a period as chief engineer for the Chattanooga Flood Protection District and as principal engineering assistant to the chief consulting engineer, TVA. He is now chief construction engineer with Holway and Neuffer.

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VOLUME 9 NUMBER 9

September 1939

Entered as second class matter September 23, 1930, at the Post Office at Easton, Pa., under the Act of August 24, 1912, and accepted for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

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Printed in the U. S. A.

CIVIL ENGINEERING

Published Monthly by the

AMERICAN SOCIETY OF CIVIL ENGINEERS

(Founded November 5, 1852)

PUBLICATION OFFICE: 20TH AND NORTHAMPTON STREETS, EASTON, PA.

EDITORIAL AND ADVERTISING DEPARTMENTS:

33 WEST 39TH STREET, NEW YORK

This Issue Contains

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SUBSCRIPTION RATES

Price, 50 cents a copy; $5.00 a year in advance; $4.00 a year to members and to libraries; and $2.50 a year to members of Student Chapters. Canadian postage 75 cents and foreign postage $1.50 additional.

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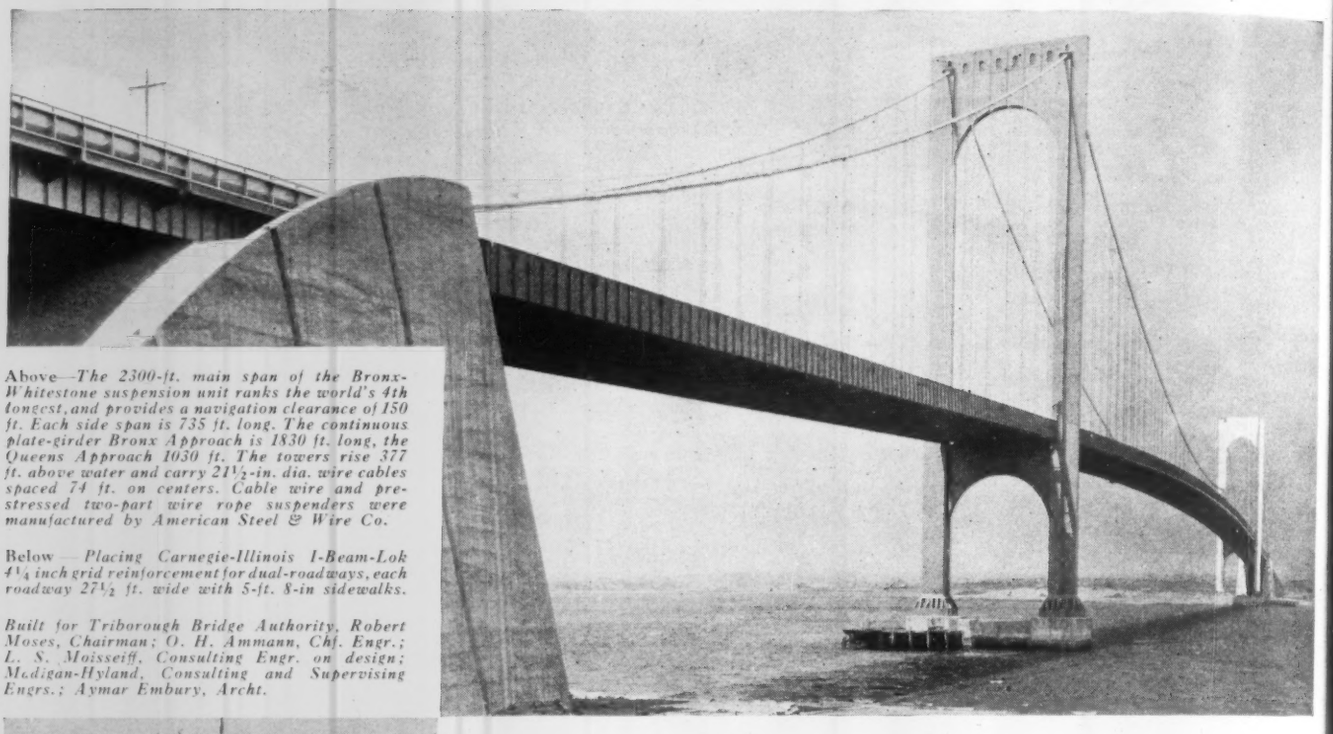
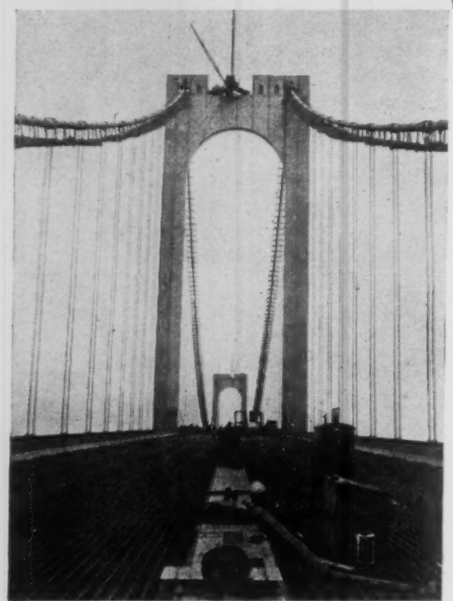
—The Bronx-Whitestone Bridge



Above—*The 2300-ft. main span of the Bronx-Whitestone suspension unit ranks the world's 4th longest, and provides a navigation clearance of 150 ft. Each side span is 735 ft. long. The continuous plate-girder Bronx Approach is 1830 ft. long, the Queens Approach 1030 ft. The towers rise 377 ft. above water and carry 21½-in. dia. wire cables spaced 74 ft. on centers. Cable wire and pre-stressed two-part wire rope suspenders were manufactured by American Steel & Wire Co.*

Below—*Placing Carnegie-Illinois I-Beam-Lok 4¼ inch grid reinforcement for dual-roadways, each roadway 27½ ft. wide with 5-ft. 8-in sidewalks.*

Built for Triborough Bridge Authority, Robert Moses, Chairman; O. H. Ammann, Chf. Engr.; L. S. Moisseiff, Consulting Engr. on design; Madigan-Hyland, Consulting and Supervising Engrs.; Aymar Embury, Archt.



THE Bronx-Whitestone bridge, spanning the East River, is the key structure in a 5-mile express highway connecting the Hutchinson River Parkway with Long Island's high-speed traffic network. It constitutes one of the most important links in the Circumferential Parkway system which, eventually, will completely girdle New York City. For travelers between the New England states and the New York World's Fair on Long Island, it affords a traffic by-pass skirting New York City and saving a distance of some 8 miles.

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UNITED STATES STEEL

Something to Think About

A Series of Reflective Comments Sponsored by the Committee on Publications

Is Diversion a Menace or a Myth?

By HAWLEY S. SIMPSON
MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
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CURRENT propaganda against the evils of diverting special highway revenues for other than highway purposes, culminating in restraining legislation by the federal government and prohibitory constitutional amendments in several states, has reached such a pass that the average motorist has become convinced he is being subjected to highly discriminatory taxation. In view of the widespread agitation against diversion, fostered by those who believe that it is a national menace, it is startling to find such a coldly analytical document as the 1937 report of the New York State Tax Commission stating that "diversion is a myth, created by the unfair use of statistics. . . ."

Taxes for Highways Only?—In attempting to separate the chaff from the wheat of this argument, the first stumbling block is to find an accepted definition of "diversion." In general the word is taken to mean using state gasoline tax revenues for purposes other than construction and maintenance of state highway systems, rural roads, and city streets, and such related functions as policing state highways. More broadly applied, it refers to such other uses of any special motor vehicle taxes, whether state or federal. Speaking historically, the avowed purpose of the first state gasoline tax, conceived in Oregon in 1919, was to supplement funds derived from general taxation for street and highway construction and maintenance. In short, gasoline tax revenues were to be devoted solely to highway purposes.

To make the arguments against diversion valid, it must be assumed that these revenues, and other special motor vehicle taxes, are sacred to the use of highways. But the U. S. Supreme Court does not concur in this view. When it was charged that a Georgia special tax violated the commerce clause of the Constitution in that the tax levied on the plaintiff company was used solely on the rural postal roads, over which the company did not operate, the Court held that: "The exaction is not to be deemed offensive to the commerce clause merely because the state, in the conduct of its fiscal affairs, chooses to use part or all of the proceeds for purposes other than the construction, improvement, or maintenance of its highways."

That there has been a substantial diversion of state motor vehicle tax revenues cannot be questioned. As states began to realize that here was an easily tapped source of funds for financing essential state functions other than highways, the practice increased until in 1937 over 160 million dollars was diverted. This represented 14 per cent of all state gasoline taxes, registration fees, and special motor taxes, after deducting administrative expenses.

Expenditures vs. Costs.—Since one-fourth of this sum was diverted in New York State, on what grounds can the State Tax Commission claim that diversion is a "myth"? To support its case, the Commission shows that total special automobile taxes collected by the state and federal government in New York for the period 1929–1937 amounted to 860 million dollars. This the Commission compares with the 1,600 million dollar expenditures for all streets and highways in the state over the same period. Hence its conclusion that "motorists have received nearly $2 for every $1 contributed by them," and so "diversion is a myth."

Above the quibble over definitions, the real issue is: Are motor vehicles now paying their fair share of street and highway costs, plus an equitable contribution to the general costs of government? In 1937 special taxes levied upon the motorist directly and indirectly, after deducting known administrative costs, totaled about 1,500 million dollars—1,153 million dollars by the states and 345 million by the federal government. In the same year, estimated expenditures for all city street and rural highway construction, maintenance, and bond service totaled 2,170 million dollars.

The question now arises as to whether these out-of-pocket expenditures represent true "costs." Since state and federal accounting merely compares capital inflow and outflow, true "costs" will be higher than annual expenditures if we use up our highways faster than we pay for them. The reverse will be the case if we build faster

than we wear them out. In the long run the two sums must be equal. Hence, for the present purpose it is sufficient to consider highway expenditures as "costs."

To Find Fair Distribution.—All who have studied this problem agree that general taxation should bear a portion of street and highway costs, yet there is no unanimity of opinion as to how to reach an equitable distribution. The Oregon State Highway Commission, after a comprehensive survey, concluded that user taxes in that state should approximate 43.5 per cent of total street and highway expenditures, while a railroad report estimates that, for the period 1921–1932, motorists in the United States should have paid 75.6 per cent of such costs. These are the two extremes; estimates by others fall between these figures. Applying the underlying data on which these percentages are based to the year 1937, the Oregon estimate (not intended for national application) indicates that the motorist was overcharged about 525 million, while the railroad report shows a subsidy to the motorist of about 220 million.

Examining the railroad figures further, it is found that the part of highway costs which it is claimed should be borne by others than users, is identical with per capita street and highway costs in 1904, before the automobile influence. These amounts are $1 per capita for rural highways and $4.20 per capita of urban population for city streets. Expenditures beyond these figures, the authors of the report conclude, were incurred exclusively on behalf of motor vehicles and should be met out of special motor imposts.

Motorist Now Pays Fair Share.—It is proper to question whether this is a sound basis for calculation. Standards of living have increased measurably since 1904. Would the public now be satisfied with the mud roads of 35 years ago? Nothing else that was acceptable to us in 1904 finds much favor today. Furthermore, highway construction costs have risen greatly in this period; thus the highway dollar today buys less comparable roadway than at the turn of the century. So we may reasonably expect that the public now would be willing, and would have to spend per capita many more present-day dollars than in 1904 to provide a satisfactory street and highway system even if there were no automobiles. If we assume a per capita expenditure even twice that in 1904, general taxation for highway purposes should have contributed in 1937 about 820 million dollars. The motorist's fair share of 1937 expenditures would then have been about 1,350 million. This is about 150 million dollars less than the total of special motor vehicle imposts in that year.

Apparently, then, the motorist is being neither unfairly treated nor subsidized. It cannot be said, on the basis of data available, that he has failed to pay approximately all, if not more than all, of street and highway construction and maintenance costs that are fairly chargeable to the users.

Autos Contribute to General Governmental Costs.—In the nation-wide investigation of automobile taxation conducted by the Bureau of Public Roads for 1932, it was shown that 17 states exempted motor vehicles entirely from taxation as personal property. About 46 per cent of all vehicles in the country were registered in those states. Of the remainder, nominally subject to taxation, only 60 per cent actually appeared on the tax rolls and paid about 36 million in taxes.

Obviously, then, motor vehicles are not bearing their proportionate share of the general cost of government through direct taxation as property. However if, as we believe, special taxation in other forms totals more than the motorists' equitable share of highway costs, the deficiency in this regard is of no great significance.

Subsidies Once Justified.—It becomes obvious, then, that diversion is indeed a myth in the sense that the automobile user is not being greatly overtaxed, and the arguments of those who set up this straw man seem to have little validity. If the motor vehicle is not paying more than its proper share of highway and governmental expenditures, and so long as state and city highway programs are formulated on sound economic bases, it matters little whether taxes are earmarked for special purposes or wholly diverted to the general fund.

This is not to say that in the past there have not been substantial subsidies through general taxation for the benefit of motor transportation. Admittedly there have been, but this is now water over the dam. In this respect the automobile is not alone, for no important form of transportation has grown to maturity without governmental subsidy. In the present instance the public demanded better roads and was willing to pay for them without giving too much consideration to where the money came from. Of much more importance than propaganda against diversion, which has been successful only in directing public attention to a controversial issue without offering any constructive solution, is the matter of a more scientific and continuing determination of highway benefits and a proper allocation of costs to benefited groups.

Financing the Highway Program.—Without question there are inequalities in the present motor vehicle tax structure, certain classes being overtaxed and others undertaxed. Similarly, inequalities exist in the distribution of taxes between users and others in various jurisdictions. Their adjustment is a real problem, to which those interested in sound highway policies, and engineers in particular, should direct attention so that the matter can be taken out of the realm of emotional controversy and legislative maneuvering and given the technical attention it deserves. Allied to this is the importance of carrying forward the development of a proper highway system equitably planned and at a total cost to the public commensurate with its value.

One important danger in the narrow viewpoint of the anti-diversionist is often overlooked. Through prohibitory constitutional amendments and otherwise, a false impression has been conveyed that motor vehicle imposts are fully adequate to meet all street and highway costs. What, then, is to prevent the passage of retaliatory amendments prohibiting the use of any taxes, other than those levied on the motor vehicle, for street and highway purposes? Should this occur, untold damage would be done to highway systems and to the motor transportation industry; either construction and maintenance would be greatly handicapped or special taxation would have to be increased. Continuation of this dangerous campaign against so-called "diversion" can easily become a two-edged sword cutting both ways.

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VOLUME 9 NUMBER 9

Engineering Training for the Construction Field

Present Opportunities for the Civil Engineer and Means of Improving Them

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THERE is greater need today than ever before, says Mr. Ackerman, for technically trained men in the construction industry. There are also "ample indications" that that industry is not employing as many engineers as it could use to advantage, and much needs to be done to sell it on the value of technical assistance. Mr. Ackerman here outlines the "basic enterprises of construction" served by the engineer, and offers valuable suggestions for broadening the opportunities. This paper was on the program of the Society for the Promotion of Engineering Education at its annual convention in June 1939.

TODAY the trend in the construction industry is definitely toward a more scientific approach. The planning of a construction program is no longer left to the quick judgment and instinct of the former "rule-of-thumb" type of construction superintendent; it is becoming, instead, a matter of precise engineering analysis. At the same time, new and larger equipment is constantly being developed which is opening opportunities for the construction of projects that would have been infeasible only a decade or two ago. In consequence there is greater need today than ever before for the employment of technically trained men in the construction industry. It is the objective of this paper both to point out the possibilities for engineers in this field, and to suggest means whereby they might be still further enlarged.

Without seeking to discuss here the "state of the nation," a brief generalizing of present conditions, as they affect construction, seems in order. During the past few years we have experienced some very fundamental changes in our national economy. First, we are gradually realizing that "unemployment" is being recognized as a permanent institution for which our government has assumed the relief burden. The second fundamental change is the general unionizing of labor, which has been accomplished in part by new forms of legislation and in part by the inducement of securing higher wages through organized effort. A third fundamental change, largely an outgrowth of the second, is a new trend in mechanization at a very high rate and, at present, not fully recognized. This mechanization is the inevitable result of an arbitrary and heavy increase in the scale of wages, which has upset the delicate balance between machine production costs and labor costs. In consequence a new ratio between these two costs becomes economically feasible and results in the introduction of new mechanical methods with a corresponding reduction in necessary labor. For example, in the case of a product formerly having a machine time cost of $10 and a labor time cost of $10, let us assume that a new wage scale makes the labor cost $15. Instead of operating at a new total cost of $25, a manufacturer can immediately afford to install a labor-saving machine with a machine cost of $12 and a labor cost of $11, or even less, and competitive conditions will force him to do so. After this the 26 per cent reduction in labor time is permanently lost.

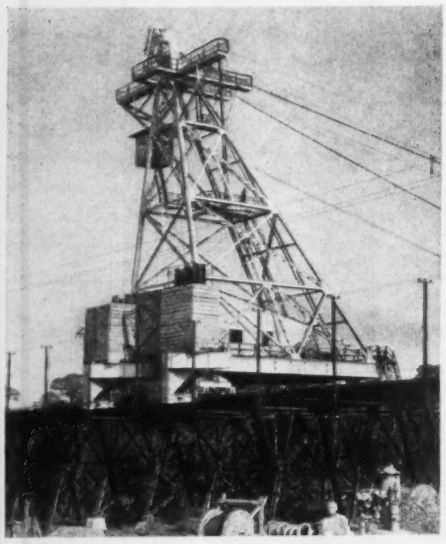
This reduction in labor, in turn, is bound to react unfavorably with respect to general employment, thus adding to the first problem unless counteracting influences are developed in the form of new opportunities and new fields of enterprise.

During this period of great change which we are now experiencing, construction activity has by and large been the expedient used to alleviate unemployment conditions, and in many cases the kind of construction has been of secondary importance, just so long as it was construction. Too much of it has been an "expedient" rather than the "means to an end." Levees, flood walls, flood control dams, and public buildings have been constructed in great numbers—but what is needed is more of the productive type of construction that will result in new means of livelihood and new opportunities for the inhabitants of a region. In recent years there have also been, of course, outstanding examples of this type of construction. For example, the All-American Canal in California, requiring the excavation of about 65,000,000 cu yd of earth, will provide permanent irrigation to some 1,000,000 acres of land and will eventually provide a self-sustaining form of agricultural activity in the place of a hopeless lack of opportunity from submarginal land. Such new developments can be achieved to an increasing extent only as the costs of construction are brought down.

Much has been said of late of the need for, and the possibilities in, a large program of low-cost housing. The general principles of such a program apply also to all other fields of construction; namely, as the costs of construction are brought down in a given field, the volume of such work goes up rapidly. Statistics compiled by the U. S. Bureau of Public Roads readily demonstrate this point. The average unit price bid on excavation in 1938 was 45.7 per cent of that bid in 1923,

even though the wages paid for common labor were 47.4 per cent higher in 1938 than in 1923, and the quality of work demanded today is considerably higher. This is largely due to the use of better equipment and more scientific management, or, in other words, the increased productivity per man. Where in 1920 one man with a four-horse team and 1/3-yd Fresno could haul earth a distance of 400 ft at the rate of 4 yd per hr, today one man with a tractor and a 13-yd scraper can move earth the same distance at the rate of 115 yd per hr.

This increased productivity per man also means that lower costs are making economically feasible new projects that were considered fantastic dreams a few decades ago. Here again the All-American Canal provides a good example. The vast amount of earth to be moved, the large amount of equipment and teams needed to perform the work, and the high cost of doing the job, at one time seemed insurmountable, but the construction industry contributed to making this canal a reality by developing its earth-moving technique to a point where, with a relatively small number of men and with machines of tremendous proportions, it was possible to dig the canal at an average cost of 10 cents per cu yd, whereas about 20 years ago the standard price for such work was considered 35 cents per cu yd.



STRUCTURAL AND MECHANICAL ENGINEERING PROBLEMS ENCOUNTERED IN CONTRACTING ARE TYPIFIED BY THIS CABLEWAY TOWER AND STEEL TRESTLE RUNWAY

As previously stated, the construction industry is confronted with a challenge to help advance the feasibility of new sources of productive enterprise and opportunity, such as irrigation projects, power developments, drilling and mining projects, new railroad and waterway facilities, factory buildings of all types, and similar enterprises. The need for new construction and employment opportunities, through lower costs and new methods, also applies of course to such basic enterprises as the construction of cross-country superhighways, the building of subways and tunnels, realinement of railroads, and the extensive reconstruction of cities to meet the traffic demands of tomorrow.

HOW THE ENGINEER SERVES CONSTRUCTION

In all these activities the engineer plays an important part because technical knowledge and skill are fundamental necessities. The basic enterprises of construction served by the engineer may be briefly enumerated as follows:

Contracting and building
Equipment and machinery manufacture
Selling of construction machinery and services
Supervision of construction for technical adequacy
Supervision of "Public Work" and "Relief" construction
Education and training of qualified personnel

A brief recital of past procedures in construction is of interest in considering the present trend and future possibilities. In the so-called "heyday" of contracting, most contractors had personal contacts, or a clientele, and secured their jobs on the basis of cost estimates that were liberal enough to assure completion by one way or another. The superintendent on the job was a king in his own right, who frequently determined to build the job in his own way, with all the temperamental stubbornness of a prima donna and with an overabundance of driving power to get the most out of the backs of men and animals. If he was accustomed to building a job with derricks, he would have plenty of derricks standing around. If his background had been developed on cableway jobs, he would want to do everything with cableways. If he was a dragline man, the only way excavation could be done would be by dragline.

As the larger and more complex construction operations came into the picture, he began to depend on the engineer to design suitable equipment and methods best fitted for the job and for developing a higher degree of planning with well-defined step-by-step procedures. After the World War, factory methods of construction and control of operation began to take hold, and today we find that a job of any importance is carefully planned in great detail. Excavation operations are worked out on models and studies made with different classes of equipment to assure the most economical procedure; handling of rivers or other hydraulic problems are planned in advance on models; equipment selection is made after a careful survey of all available types, and frequently it becomes necessary to design new equipment of a type not on the market because a careful engineering analysis has disclosed a cheaper and better way to do the job.

ENGINEERING ECONOMICS IN PLANT DESIGN

Construction operations are largely electrified, and the planning of a power system and suitable controls and interlocks between operations is often a major piece of engineering. The modern construction superintendent is alert to the necessity for having competent technical advice in the selection of safe equipment and of adequate capacity; correct selection is no longer a question of snap judgment. Many new types of equipment employ special alloys, improved types of welded construction, and metallurgical developments which must be properly related to the work in hand, and the engineer must be able to judge the merits of such features and to make improvements as the work progresses. It has become well recognized that a great many large construction operations can be done more economically provided enough money has been spent at the start on the equipment and construction plant. The first cost of equipment is generally a small proportion of the running costs of operation, maintenance, and repairs. A $50,000 shovel may do $500,000 worth of work. The purchase of a $60,000 shovel with better mechanical design and metallurgical features may readily prove a wiser investment, whereas the purchase of a $40,000 shovel, while an apparent economy at the start, may require a greater amount of repair and maintenance work, with consequent greater outage time, all tending to eat up most of the profit.

This same basic principle must be considered for every operation on a job, and the determination of a proper balance is a complex engineering problem which must be correctly solved at the start, lest the job be a failure. Once an unsuitable type of construction plant has been erected, it usually costs more to change or replace it than to complete the operation on an uneconomical basis.

After a large plant is erected or installed, the training of a construction personnel to operate it is frequently the job of the technically trained man.

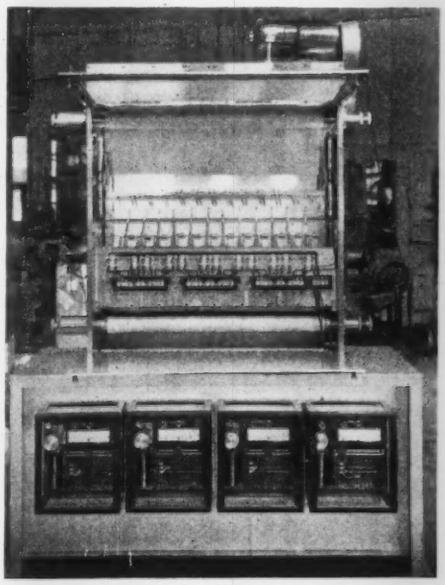
THE IMPORTANCE OF COST CONTROL WORK

During the period of construction there are opportunities for technically trained men to carry on cost control observations that will pay large dividends. Engineering cost keeping is not intended for developing historical records, but for analyzing in detail the cost make-up of various current operations and just where improvements can be made; this requires close contact with the work, a full understanding of job requirements, and ingenuity in finding improvements. Such analyses apply also to timekeeping of construction equipment, just as has been practiced for years in timekeeping of employees. This can be very productive because, while an employee's time may be worth only $8 or $10 per day, a large construction machine may cost $50 or $100 per day, and economies and improvements in operation are correspondingly greater.

The technically trained man can introduce various types of recording meters of the type employed for years in power stations or manufacturing plants. For example, such meters are now used for accurately recording the quality of the concrete coming out of a mixer, or for recording the productivity of various types of crushing equipment so that the metallurgy of important wearing parts can be improved as the need arises. The performance of all types of transporting equipment can be measured by recording apparatus, and operating cycles made up on the most economical basis. Recordings of electric power consumption of entire construction operations will disclose, among other things, where drills, compressors, or other equipment are being shut down too early for lunch or started fifteen or twenty minutes after lunch. All these recording devices provide important means for maintaining a job control, provided they are subjected to intelligent analysis.

Another field of activity which the technically trained man can enlarge to advantage is in the detailed measuring of recurring cycles of operations, such as hauling, drilling, or form handling, by recording actual performance and determining where a rearrangement of equipment will result in a more economical procedure. The supervising foreman or superintendent cannot afford the time for such detailed attention. It is surprising how little is known about the ability and performance of construction equipment, and the judicious use of carefully secured operating data offers widespread opportunities for improvement. Such observers must have good judgment, an understanding of management problems, and considerable diplomacy in dealing with the workmen under observation.

The engineer also has an opportunity to play an important part in labor relations. We have had ample evidence of late of the importance of keeping enterprises on a productive basis, and that can only be done if labor relations are on a sound and fair basis. Unquestionably, organized labor and collective bargaining are here to stay, and the sooner an intelligent understanding on the part of employers, and a qualified type of leadership on the part of labor become more general, the quicker a stable relationship will follow. With this in mind, it would be highly desirable for undergraduate engineers and young graduates to experience some of the problems of labor. It is begging the question to contend that it is beyond professional dignity to participate in labor organizations and in collective bargaining. While this may be so in professional work, nevertheless every supervising engineer is confronted with the problem of dealing with organized groups, and he can do this more effectively if he, himself, has experienced the promoting, organizing, collective bargaining, and electing which now, to a large extent, dictate the future of a workingman's life. The apprentice engineer who takes an interest in these activities and works at a trade for a year or so can, by virtue of his education and training, make some valuable contributions toward a sound labor policy, and at the same time gain experiences of inestimable value to him in later years.



PRECISE CONTROL OF CONCRETE PRODUCTION CALLS FOR ENGINEERING SKILL

The Automatic Graphic Recorder Shown Here Was Used at the Central Mixing Plant on the Marshall Ford Dam to Record Rate of Output, Weights of Raw Materials, Consistency of Concrete, and Related Data

WIDE OPPORTUNITIES FOR DEVELOPING NEW TOOLS

Hand in hand with the advances in construction operations and methods is the improvement of construction equipment and the development of new devices. Equipment in the construction field becomes obsolete at a very fast rate, and redesign is almost a daily occurrence. Furthermore, the pioneering advances in equipment design which made feasible such projects as the All-American Canal are bound to have a marked influence on the economic feasibility of many future enterprises. New alloys and new metals will improve the life of equipment and double their speeds of operations. Here is a field for the ingenious and inventive type of engineer who can combine design ability with a practical approach.

There are wide opportunities for developing new construction tools and methods. In the drilling of rock, for example, much has been accomplished in developing higher speeds and lower costs, yet some are claiming now that unheard-of speeds will eventually be developed by the use of piercing, high-temperature flames. On the other hand, some very crude procedures are still in use today because no better methods have been developed. For example, in excavating a deep shaft into rock, the method of muckingout the rock is no different from that employed in the Middle Ages; namely, a man stoops down and picks up the rock and puts it into a bucket. Nobody has succeeded in developing a device for improving this back-breaking operation.

There has been very little done in the field of merchandising construction services, and some important opportunities may be developed in this respect. The industry is confronted with the most severe type of competition because there is a general attitude that almost anyone can undertake construction work. When new work is under consideration, many individuals or agencies are disposed to employ their own superintendent and construction organization. This heavy competition from day-labor work by governmental

and other agencies has prevented the construction industry from maintaining a stable basis and protecting its economic status. Efforts to improve this situation must be supplemented by intelligent readjustments within the industry, but in any case there is a large amount of work to be done in this general field.

In the field of merchandising construction equipment, important new opportunities are developing. Many equipment manufacturers have local sales representation, and one sales representative may serve eight or ten different manufacturers. The highly technical developments that have occurred in recent years have, in many cases, prevented such representatives from keeping up with progress, and their function has reverted to notifying the manufacturers when a new job is coming up so that they can send their factory experts to confer with prospective customers. If sales representatives were able to furnish competent technical advice on short notice to their local customers, their usefulness would be very much more effective. There is considerable opportunity for alert young engineers to get into this field to supplement the business skill of the representatives with adequate technical information gained from intensive contact with construction and manufacturing activities.

The engineer-designer of a project can help to advance the art of construction if he makes it his business to understand the limitations under which construction equipment operates, as well as its possibilities of performance. If he familiarizes himself with the practical solution of a problem, and participates in the development of new equipment for a project, he can often help to raise the general level of accomplishment.

All too often there is an attitude that engineers are too theoretical and not practical minded, and this view is held with arbitrary generalization by many contractors. Some designers seem to display a serene confidence in the ability of the field man to employ "sky hooks," and to do things in an impossible sequence, or within space limitations entirely out of reason; when in addition to this the field engineering and inspection is founded on an arbitrary adherence to drawings and specifications, regardless of the dictates of good judgment, it is understandable why some contractors have been disposed to question the usefulness of engineers in their own businesses. (Room for improvement here is of course not limited to one side.)

DIFFICULTIES OF THE YOUNG INSPECTOR

One of the difficulties with much construction inspection is that frequently inexperienced young engineers, without suitable training for the work, are given assignments of supervising and inspecting important construction operations. They are expected to read the specifications and to see to it that the contractor performs accordingly. Sound judgment, which would help to promote a better job and better feeling and confidence among the various parties, is missing in too many cases. Most young inspectors are not trained to realize the important responsibilities that rest on their shoulders. In consequence, their failure to act at critical moments results in many things being left undone—and there is no means of evaluating the consequences of such inaction. The subjects of supervision and contract administration deserve further attention in engineering training.

If we are prepared to accept the principle that in the future a certain percentage of our population will be on "relief" rolls, and will be assigned to "relief" construction projects under the supervision of publicly employed engineers, then a logical corollary to that principle dictates the need for a new type of supervisory personnel for such work. As already stated, most "relief" work is now an expedient. There is no future in it and no inducement to stimulate a high standard of performance. Some projects have been observed where the outlook of the men was no better than that of a chain gang.

In the future, intelligent supervision of "relief" work must include an understanding type of leadership which is designed to rehabilitate the workers, develop a new outlook for them, and train them in some trades or higher degree of skill to widen their usefulness for other gainful enterprises.

CONSTRUCTION COURSES IN ENGINEERING SCHOOLS

To round out the needs of the construction industry, there is also the all-important function of educational institutions in developing competent engineering and supervisory personnel (and here the trade schools also deserve mention). In many engineering curricula the so-called "construction subjects" are only a gentle introduction to the field. If an effort were made to develop an engineering course devoted to the field of construction, the following subjects are suggested in addition to those common to the various branches of civil engineering:

- Job management
- Bonding and financing
- Estimating and job planning
- Contract law
- Labor relations and laws
- Ethics
- Personnel training and welfare
- Safety
- First aid, health, and workman's compensation
- Accounting and cost keeping
- Equipment design
- Measuring equipment performance
- Concrete and aggregate production and handling
- Earth and rock handling
- Electric motors, controls, and power distribution

It would also be highly desirable to encourage students who are interested in construction to expand their courses and undertake some very practical types of study. For example, a thesis or seminar work could be undertaken on a grade separation or similar moderate-sized public project at the time it is currently being considered. The students could prepare their own estimates, formulate a program of construction, equipment, and labor requirements, and later compare their work with the official bid prices and the execution of the work by the contractor. By such direct comparison they would acquire a good understanding of the contractor's approach to such a problem.

The enlistment of active leaders in the construction industry for lecture purposes, and the encouragement of builders to record their wealth of experience for reference or textbook purposes, also bear a natural relationship to this program.

In 1928 construction contracts amounted to 10½ billion dollars. In recent years we have had less than half this volume, and this mostly in public works. There is, however, every reason to believe that some day we shall see the 1928 figures exceeded by a considerable margin. Construction is one of the major branches of engineering and of business and as such deserves an important position in engineering curricula.

There are ample indications that the construction industry is not employing as many engineers as it could use to advantage, and much needs to be done to sell the value of technical assistance to the construction industry. A cooperative study of some of these problems by such organizations as the Society for the Promotion of Engineering Education, the American Society of Civil Engineers, and the Associated General Contractors of America, together with equipment manufacturers, should develop important suggestions in planning for the future.

The Pensacola Project, Oklahoma

Multiple-Arch Dam, Now Under Construction, Is First Unit in Development of Grand River

By E. Lawrence Chandler

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HALF-WAY point in concrete pouring has been passed on the Pensacola Dam, the mile-long multiple-arch structure now under construction on the Grand River in northeastern Oklahoma. In these pages Mr. Chandler gives a brief general description of the project, and then takes up in more detail the methods employed in construction. Of particular interest are the preparation of foundations for the arches and buttresses, and the highly flexible arrangements for handling forms and concrete.

HYDROELECTRIC development at or near the site of the Pensacola Dam on the Grand River in northeastern Oklahoma has held the attention of a succession of individuals and organizations for more than forty years. The objective of this long period of planning and effort began to take form with the creation of the Grand River Dam Authority in 1935 by the Oklahoma State Legislature.

The Grand River Dam Authority is an agency of the State of Oklahoma, composed of 14 counties in the northeastern part of the state. It is vested with broad powers for purposes of power development, flood control, and a general conservation program. The Pensacola project is the first and major unit in the Grand River development. Ultimate completion of the program will call for two lesser dams at the Markham Ferry and Fort Gibson sites downstream.

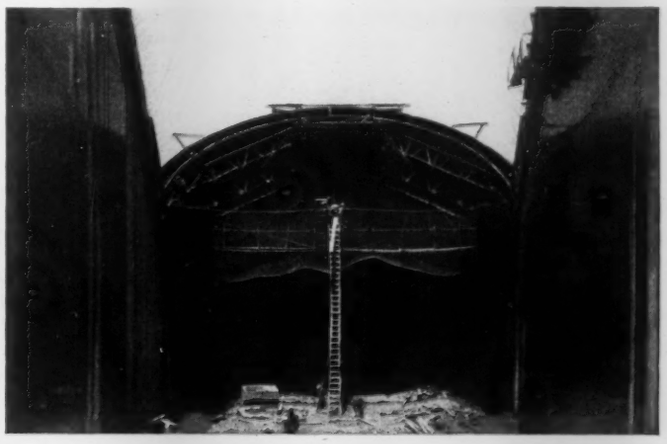
Pensacola Dam provides both flood control and power. With a drainage area of 10,415 sq miles tributary to the reservoir, the largest flood of record is 215,000 cu ft per sec. A spillway capacity of 525,000 cu ft per sec is being provided, although it is planned to control all except extreme floods to a maximum of 80,000 cu ft per sec, which is about bank-full capacity of the river below the dam. The large amount of storage in the reservoir (1,680,000 acre-ft) will provide a regulated flow averaging 3,000 cu ft per sec.

Annual power production of 200,000,000 kwhr is anticipated, with an average operating head of about 115 ft. This output will be doubled when the two downstream plants are completed. Normal tailwater will be at El. 620 and the operating pool level will be maintained at El. 745 as much of the time as possible. Maximum reservoir level has been planned as El. 750, although a possible modification requiring raising to El. 755 may be accomplished without any changes in the dam.

The general features of the dam are to be seen in Fig. 1 which is a view looking southeasterly on June 17, 1939. The dam proper consists of about 5,680 ft of continuous concrete structure, the outstanding portion of which is the multiple-arch section composed of 51 arches extending easterly nearly 4,300 ft from the west abutment. Each arch has a clear span of 60 ft, buttresses being of the hollow type, spaced 84 ft center to center. An interesting feature of buttress design is found in the arrangement of joints. As may be seen in the picture, each buttress is composed of a number of separately poured monoliths so arranged that joints at right angles to the axis of the dam incline upward with a slope of 1 on 4 toward the downstream end of the buttress. Monoliths are 15 ft high measured at right angles to the joints mentioned, and their lengths vary so that the joints between them are staggered. This design is considered to have definite advantages over others which have introduced joints of irregular form and difficult construction in an effort to minimize cracking along the lines of principal stresses. Buttress tops are at El. 657, the largest buttresses being about 150 ft high. Arch rings are poured in 15-ft lifts measured along the slope of the arch.

Extending easterly from the multiple-arch portion is the main spillway, which is a gravity-type structure. It will be 861 ft long, with crest elevation at 730, and will be surmounted by 21 Tainter gates, each 36 ft long by 25 ft high. The partially completed structure may be seen in the distance in Fig. 1. Discharge from the

The Gravity Spillway of Pensacola Dam
Several of the Blocks Are Already at Full Crest Height; Tainter Gates, 36 Ft Wide and 25 Ft High, Are to Follow



One of the Arches During Construction
Note the Safety Net, Which Moves Upward with the Intrados Form

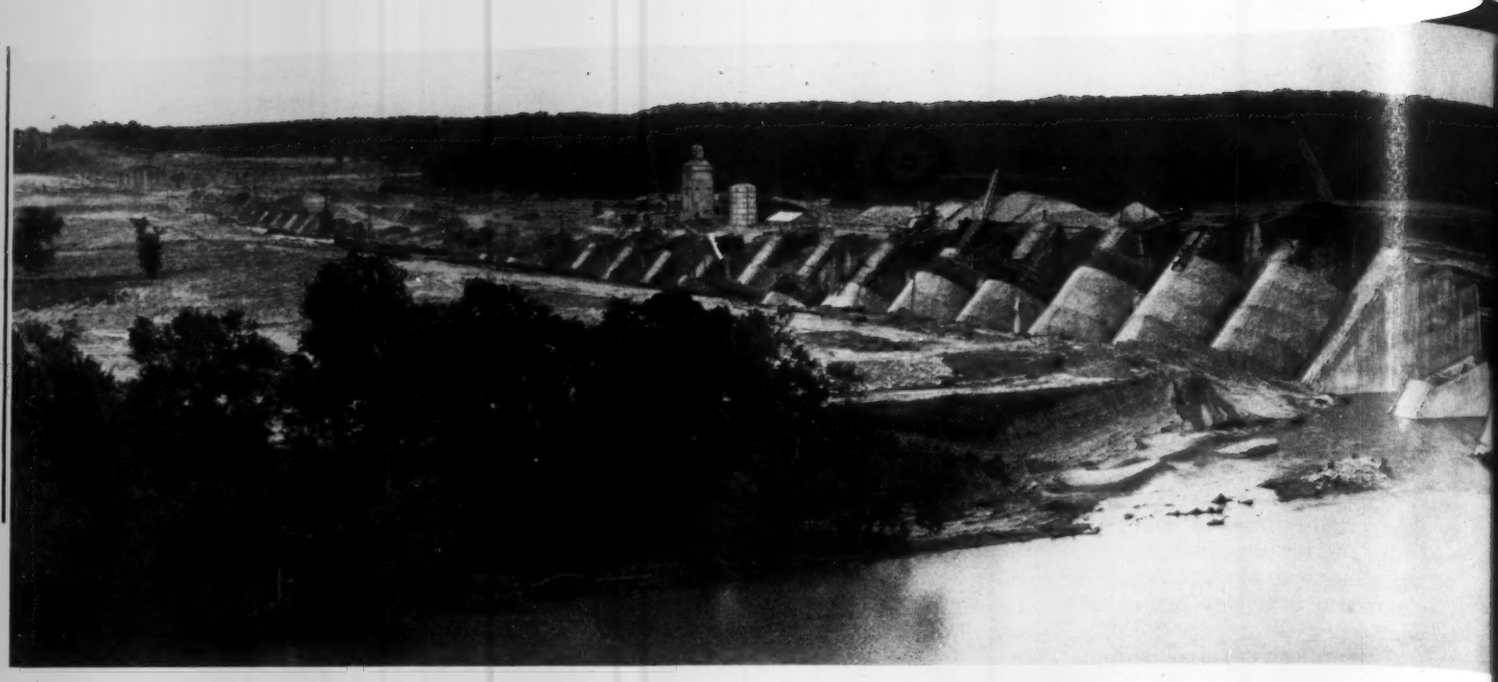


FIG. 1. PENSACOLA DAM—VIEW LOOKING SOUTHEASTERLY, JUNE 17, 1939. SECOND COFFERDAM, WITH POWER HOUSE EXCAVATION UNDER WAY, SHOWS AT THE RIGHT. GRAVITY SPILLWAY IS TO BE SEEN UNDER CONSTRUCTION IN THE DISTANCE. AGGREGATE STORAGE AND CONCRETE MIXING PLANT ARE ON THE DOWNSTREAM SIDE OF THE DAM

spillway will flow into the present river channel about a mile downstream from the dam. The light-colored strip showing in the picture along the foot of the distant, wooded hill is a dike along the spillway channel.

The east end of the dam is a gravity-type, non-overflow structure extending from the spillway into the east abutment.

About a mile east of the main dam two auxiliary spillways are located in a saddle in the hills. These are equipped with 21 Tainter gates, each 37 ft long by 15 ft high. Discharge from these spillways will unite with that from the main spillway before emptying into the river.

A 20-ft roadway will be carried across the top of both the main dam and the auxiliary spillways.

The power house is being built in the old river channel close to the west abutment. The original installation will include four 20,000-hp units and a 750-hp house unit, and emplacements will be provided for two future turbines. The switchyard is being located on top of the west bluff at an elevation about 146 ft above normal tailwater, connections from power house to switchyard being through a bus tunnel and vertical shaft in the rock of the abutment.

CONSTRUCTION METHODS

The Grand River is flashy, and records indicate possibilities of high water at all seasons so that substantial cofferdam construction is essential. The contractor determined on steel piling, cellular construction, as most suitable. In Fig. 1 is to be seen the second cofferdam with the power house excavation under way. The cells are elliptical except at corners and ends, where circular structures are used. Satisfactory service has been afforded to date. Because of a moderate rise in the river, it became necessary to flood the second cofferdam just before it reached completion, but there have been no large floods and work has proceeded without serious interruption from the river.

The first cofferdam was of construction similar to the one shown. It included the buttress which now forms part of the easterly side of the present cofferdam, and was tied in to the left bank of the river, leaving the stream in its original bed along the foot of the west bluff.

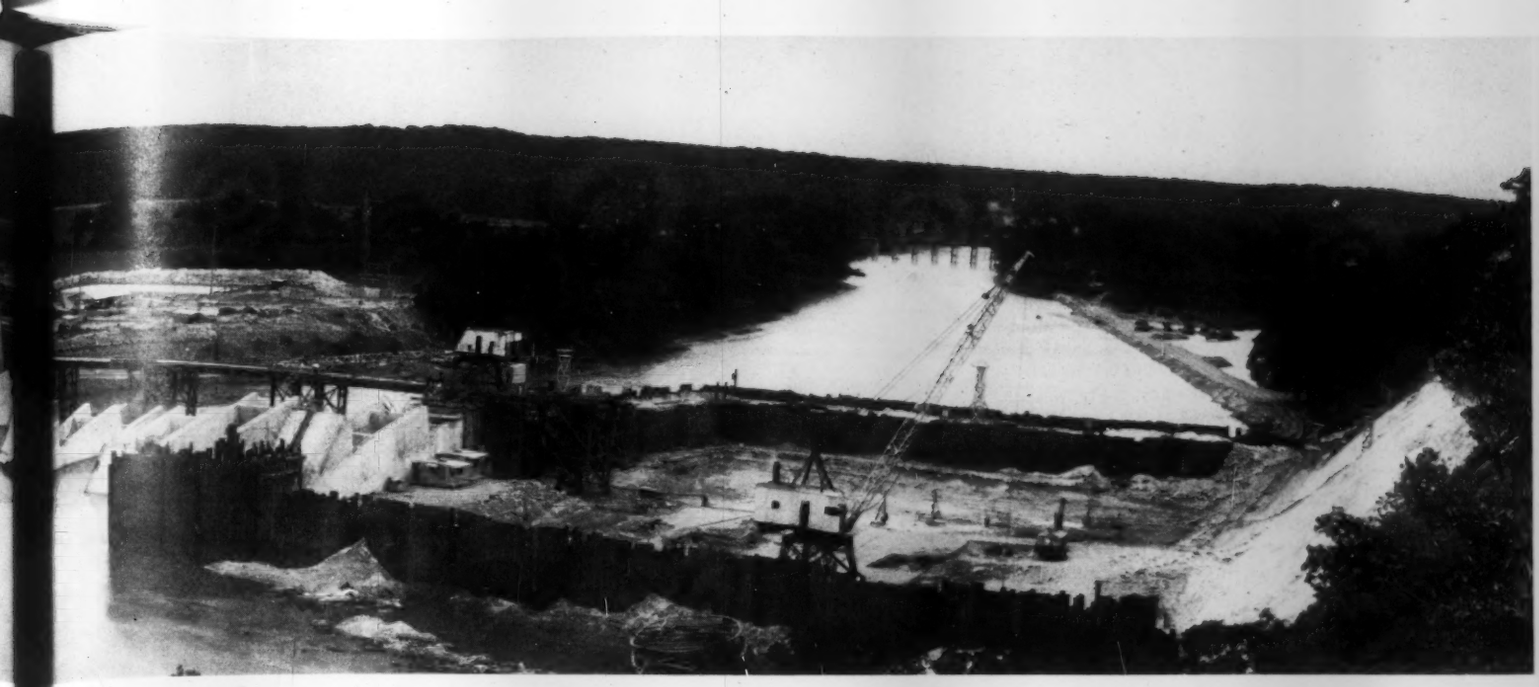
Final closure will be made through arches Nos. 7 and 8. In each of these bays a gravity-section block of concrete has already been built with three 8 by 10-ft openings. These structures are to be seen nearly submerged, with driftwood lodged on them, in the picture.

About 1,300,000 cu yd of earth excavation are involved in the dam construction, the major portion being removal of overburden from the main dam foundation. A variety of equipment has been employed for this work. Elevating graders played a prominent part in handling the fine materials across the floor of the valley, where the overburden depth ranged from 25 to 30 ft. Much dirt was also moved with 12-cu yd wheeled scrapers. Shovels and trucks have accounted for the remainder.

Preparation of the foundation, although it cannot be classed as extremely difficult, presented its problems. The dam is founded in Boone chert, one of the Mississippian formations. This rock varies erratically from layers of pure limestone to a limestone with large chert content. Near the top much of the material is in layers of from a few inches to one or two feet in thickness, with a dip approximating 1 per cent toward the southwest. Some water-bearing seams were disclosed by borings, although in general the foundation is very good. Over most of the foundation area it was necessary to excavate only a few feet of inferior surface rock to reach a satisfactory material.

Grouting under the main dam foundation is being carried to a depth of approximately 20 ft below the bottoms of the footings, and a cutoff curtain along the upstream line of the arches and buttresses is being provided to a depth of 30 ft. The underlying seams become progressively tighter as the distance from the river increases, so that portions of the area take no grout at all.

FIG. 2. EXCAVATION FOR BUTTRESS FOOTING IN SECOND COFFERDAM



The general program of foundation preparation for the buttresses and arches was to remove the top strata of inferior rock from the footing areas by mass excavation methods. The maximum depth so removed was about 10 ft, and in places excellent rock was found at the surface. Rock of suitable character having been reached, the general scheme contemplated the shooting of trenches for buttress footings—although in some instances where mass rock excavation was carried to considerable depth, and the downstream face of the excavated area provided a substantial resistance to sliding, it was deemed sufficient to provide keyways in the rock in lieu of trenches.

All the rock encountered in trench work was hard and brittle so that it was very difficult to attain, by ordinary drilling and shooting methods, the neat line desired for the arch cutoff. After considerable experimentation the contractor resorted to broaching as being the most satisfactory method of carrying out this portion of the excavation. The resulting trenches have been quite satisfactory. The character of the rock may be seen in Fig. 2, which shows conditions at a location about in the center of the original river bed. Arch cutoff trenches are to be seen in Fig. 3, a view looking toward the river from the upstream side of the dam on April 4, 1939.

Chief equipment for drilling has been wagon drills operated from a central compressor station located near the mixing plant, augmented by a number of portable compressors and by jackhammers. Excavation has been by shovel and truck haul.

Both the design and the location of the dam lend themselves to a continuous high rate of construction progress. Plant for concrete placing should, therefore, be flexible, mobile, and speedy. Final selection as made by the contractor after careful study of a number of proposed methods is indicated in Fig. 4, which is a schematic drawing showing the types and number of units of equipment employed for handling forms and placing concrete.

Fig. 3. Broached Trenches for the Footings of the Arch Cutoffs

The cranes are readily moved from place to place, are rapid in action, and have good salvage value. Trucks were decided upon as providing more flexible transportation than car haul. With trucks it is possible to spot concrete buckets within ready reach of the crawler cranes working between buttresses, as well as the larger whirleys operating at higher levels.

Coarse aggregate in four gradations is obtained from a limestone quarry operated by the contractor on property owned by the Authority. Sand comes from the Arkansas River at Muskogee. All aggregates are shipped in by rail and unloaded into storage piles by a belt conveyor system which may be seen over the top of the dam structure in the center of Fig. 1. The access railroad for the job connects with the Kansas, Oklahoma and Gulf Railroad about $4^1/_2$ miles to the west, crossing the river on the bridge to be seen downstream in Fig. 1.

CONCRETING PLANT; CONCRETE STRENGTHS

Concrete for the principal structures is mixed in a central plant located just downstream from the dam at about the middle of the valley. Three 2-cu yd units comprise the mixing equipment. From the mixers concrete is discharged directly into 2-cu yd, buttom-dump buckets on trucks and is then transported by truck to within reach of one of the placing cranes. Each truck body will accommodate two buckets; hence the empty bucket for the return trip can be placed on the truck by a crane just before picking off the outgoing loaded one.

With the arrangement of batch weighing installed at the plant and with the mix designs in use, it is possible to pour at the same time in the gravity portion of the dam and at three places in the multiple-arch section and power house. The best record to date for an 8-hr shift has been 1,000 cu yd, although a rate of 150 cu yd per hr has been maintained for short periods.

The rather high test-strengths of 3,500 lb per sq in. for concrete in the arches and water-contact slabs of the buttresses, and 2,500 lb per sq in. for other structures, have been used as standard. These strengths have been met consistently with mixes using as low as 0.8 bbl cement per cu yd with a water-cement ratio of 0.62 by weight and 6-in. maximum aggregate, for gravity structures; 1.00 bbl cement with a water-cement ratio of 0.576 and 3-in. maximum aggregate, for buttress side walls and footings; and 1.10 bbl cement with a water-

cement ratio of 0.56 and 3-in. maximum aggregate, for the 3,500-lb concrete. These designs require careful laboratory supervision at the plant. During the earlier stages of the work higher cement contents than those listed here were used, and for part of the time lower water-cement ratios were employed. It appears that excellent results have been obtained from the proportions mentioned.

The specifications require the use of steel or steel-lined forms for the buttresses and arches. Owing to the fact that the dimensions of the bottom row of blocks on any buttress are governed by the elevation of the footings, the contractor elected to build steel-lined forms for those blocks. Above the bottom row in each buttress the forms are of steel. Because of battering walls, the forms for any particular block are used only once on a buttress, after which they are transferred to another one. This requires a large amount of form moving and still further emphasizes the value of speed and ease of operation of the cranes.

TOPPING OUT A BUTTRESS
The Top Is About 100 Ft Above the Decking; the Bottom, 46 Ft Below

Arch forms are steel throughout. Intrados forms are shifted up the buttresses on sectional track, which is supported on brackets fitted into recesses in the buttress side walls just below the spring line of the arches.

Excavation was started in August 1938, and the first concrete was poured on December 30. Buttresses and arches are now well advanced, as may be noted in the general view of the job, and the first concrete in the power house was placed June 29, 1939. With the total amount of concrete in the job estimated to be somewhat greater than 500,000 cu yd, approximately 60 per cent had been placed by August 28. Completion of the project is scheduled for the spring of 1940.

The Massman Construction Company of Kansas City has the main contract for the dam, excavation having been done under sub-contract by the Sammons-Robertson Company of Huntington, W.Va. Turbines are being supplied by Allis-Chalmers Manufacturing Company and will be installed under the main contract. Westinghouse Electric and Manufacturing Company has the contract for furnishing and installing the four 16,000-kva main generators and the 625-kva house generator. A contract for the switchyard and electrical accessories is held by the Brooker Engineering Company, Detroit, Mich.

The project is being financed under a loan and grant agreement with PWA. H. H. Ferguson is project engineer. Design and supervision of construction are being carried out by Holway and Neuffer, consulting engineers, Tulsa, Okla., under contract with the Grand River Dam Authority, and W. R. Holway, M. Am. Soc. C.E., is serving as chief engineer. Victor H. Cochrane, M. Am. Soc. C.E., is special consultant on structural features of design, and Barry Dibble, M. Am. Soc. C.E., is electrical consultant. The writer is in charge of construction.

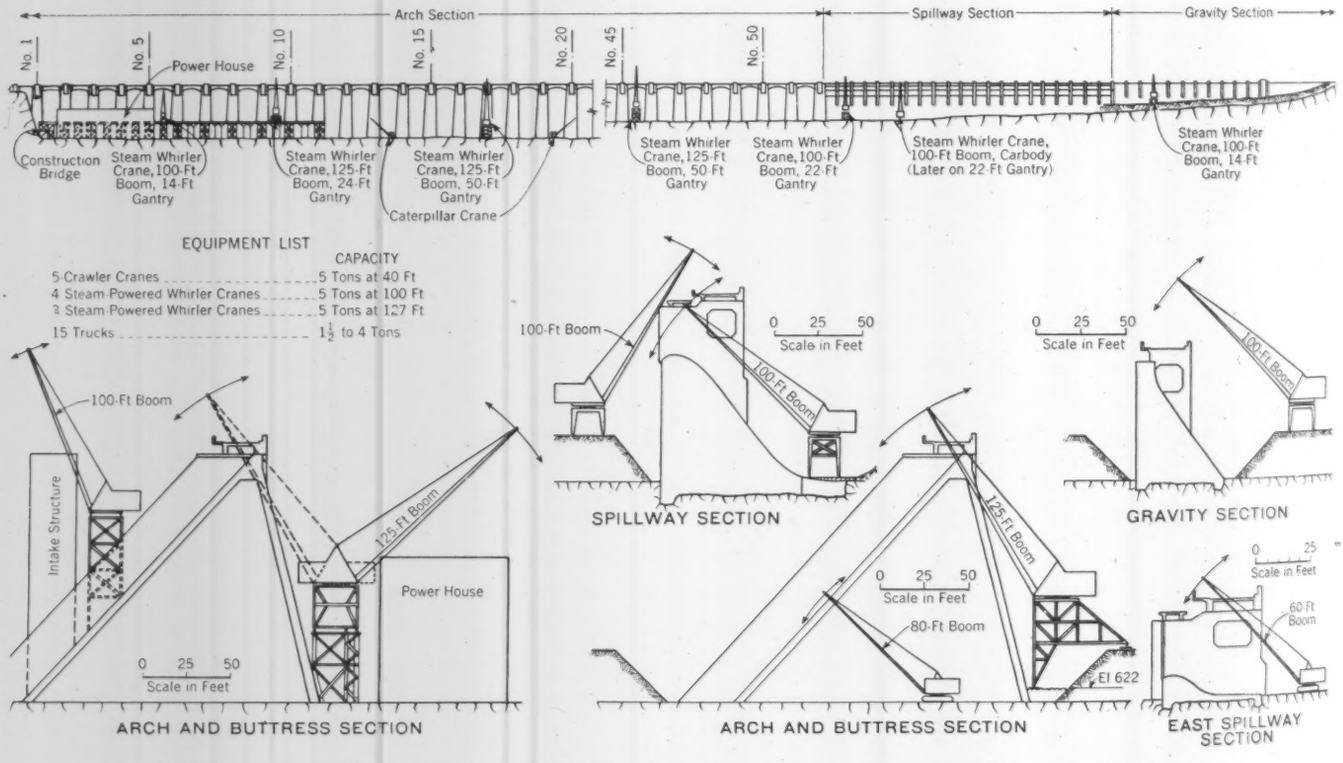


FIG. 4. SCHEMATIC DRAWING INDICATING PRINCIPAL EQUIPMENT FOR HANDLING FORMS AND CONCRETE

Measuring the Hazards on Rural Highways

Analyses of Indiana Accident Records Utilize "Frictional Classifications" to Advantage

By J. T. Hallett

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THE object of this paper is to outline a method of measuring the hazard on sections of rural highways and to emphasize the need for other and more refined units of measurement for engineers in traffic work. More than 5,500 rural highway accidents are classified on a "frictional" basis and the accident rates for different vehicular volumes and numbers of lanes are presented and analyzed. The "frictional classifications," Mr. Hallett believes, should furnish the traffic engineer with valuable data for use in design, improvement, and maintenance of rural highways; he suggests the importance of similar analyses directed to the measurement of such highway characteristics as convenience and economy.

THE traffic engineer's job is to devise ways and means to promote the convenience, the economy, and the safety of traffic, and to establish, as far as possible, the proper balance between these three characteristics. However, before he can assure himself that he knows the proper balance, and that he can obtain it at a given location, he must establish units of measurement and methods of procedure.

Traffic engineering is now going through the formative period common to all branches of engineering, during which such units and methods are evolved. In this connection much work is now being done on the various economic phases of motor vehicle transportation. Much more needs to be done.

With reference to the conveniences and inconveniences in motor vehicle traffic, very little accurate and measurable work has been accomplished other than in providing a surface over which to operate, and there is yet a great deal that should be done along this line, particularly in setting up units of measurement and establishing a "normal" or "expected" convenience. As for safety, some results have been achieved and some crude units of measurement have been established, together with expectancies. However, most of this work has been in cities, and the units and techniques do not fit the traffic conditions of rural highways.

With this last phase of the problem in mind, a study has been made of about 5,500 motor vehicle accidents occurring during the fiscal year of 1938 on about 3,500 miles of state and U. S. highways in Indiana, accidents in towns of 1,000 population and over being excluded. The results are presented herewith in Tables I and II.

In Table I, the accidents are tabulated by "frictional classifications," the significance of which will be discussed later in detail. A further division is made according to the reported principal contributing factor, giving preference to the road condition when it appeared that that factor entered into it. Finally, accidents are segregated as to severity (property damage, personal injury, and fatal) and as to time of occurrence (night or day). The amount of property damage reported and the number of fatalities and injuries are also listed.

Percentage and Severity of Various Types of Accident

"Intersectional friction" is evidenced by collisions at intersections, and accounts for 18.9 per cent of all accidents reported on these highways for the period of study. Intersectional accidents were 34.5 per cent at night vs. 49.5 per cent at night for all accidents; 2 per cent were fatal vs. 3.9 per cent fatal for all accidents; and property damage reported was $133 per accident vs. $165 for all accidents. Since only 30 per cent of state highway traffic is at night, these data indicate that driving at intersections is only slightly more hazardous by night than by day and that, in general, the ratio of night to day hazards is lower at intersections than at other places on the highways. Further, intersection accidents are less severe than the average of all highway accidents. These relations are somewhat different from those in towns and cities. The greatest contributing factor or condition is the left turn.

"Medial friction" is evidenced by collisions at or near the center of pavements by vehicles going in opposite directions, usually resulting in a head-on or side swipe, and accounts for 28.9 per cent of all accidents. Of the medial friction accidents, 55.9 per cent occurred at night as against 49.5 per cent at night for all accidents; 4.6 per cent were fatal vs. 3.9 per cent fatal for all accidents; and there was $212 property damage per accident vs. $165 property damage per accident for all. These data indicate that the hazards for night traffic are greater at the center of the road than at other places and the accidents there are considerably more severe. The medial friction accidents are good ones to try to eliminate. The greatest contributing factor is "crossing center line to pass."

"Internal stream friction" is evidenced by collision of cars going in the same direction but at different speeds, or hitting a parked car on the pavement headed in the same direction, and accounts for 28.6 per cent of all accidents. Internal stream accidents were 50.4 per cent at night vs. 49.5 per cent at night for all accidents; 2 per cent fatal vs. 3.9 per cent fatal for all accidents; and there was $133 property damage per accident vs. $165 property damage per accident for all accidents. These data indicate that the hazard per traffic unit is higher at night than by day, but is about average or normal for all accidents. The severity is less than average and the same as for intersection accidents. The greatest contributing factor or condition is "parked, slow, or stopped car on road" indicating speed differential.

"Marginal friction" is evidenced by collision with cars parked along the shoulder, poles, ditches, rough shoulders, etc., and accounts for 20.4 per cent of all accidents. Of the marginal friction accidents 50.5 per cent were at night vs. 49.5 at night for all accidents; 4 per cent were fatal vs. 3.9 per cent fatal for all accidents; and there was $150 property damage per accident vs. $165 property damage per accident for all accidents. Thus the percentage of marginal friction accidents is only slightly higher at night than the average, but considerably higher per unit of traffic than by day. The severity is only slightly greater than average but greater than intersectional and internal stream when severity is measured by percentage of fatalities. When measured by amount of property damage it is less than average but greater

than intersectional and internal stream. The greatest contributing factor or condition is "to avoid animal, pedestrian, or vehicle." A great many marginal friction accidents are the result of avoiding what otherwise would be internal stream friction.

The pedestrian accidents as a whole were only 2.4 per cent of all, but there were more "fatals" in this group than in either the intersectional or internal stream classifications. Railroad crossing accidents are only 0.4 per cent of all. The percentage of fatalities is high but the number is less than that caused by any of the other frictions. "Mechanical defects" includes only 0.4 per cent of all accidents and for the present purpose need not be given much consideration. This classification does not represent the effect of mechanical defects, but was only created for use where no other classification would apply.

A review of this tabulation, with the averages and comparisons, enables us to place a value on, or to rate, any plan or device that will reduce or eliminate any of the frictional groups of accidents. In planning construction or control devices for traffic to eliminate one class of trouble, great care should be taken to prevent creating other troubles as great or greater.

RELATION OF ACCIDENTS TO TRAFFIC VOLUME

As a second part of this study, certain relations of accidents to traffic volume and number of lanes were investigated. The same accident reports were used as in Table I, except for the omission of the "pedestrian," "railroad crossing," and "mechanical defects" groupings, which are relatively of no importance in this study. The roads were segregated into 2-lane, 3-lane, 4-lane undivided, and 4-lane divided; and the two-lane roads were further classified in 6 groups, by average daily traffic volume.

These data are presented in Table II. The sample on the 4-lane divided pavement is not so large as for other groups, and does not cover a full year for one section, but it is presented with the belief that it has some value. To some degree it represents what may be expected of such construction and enables us to give dual pavements a safety rating.

TABLE I. ACCIDENTS ON 3,500 MILES OF STATE AND FEDERAL HIGHWAYS (RURAL) IN INDIANA, JULY 1, 1937, THROUGH JUNE 30, 1938, BY TYPE

Major Types	Specific Causes or Conditions	Fatalities	Persons Injured	Property Damages	Number of Accidents: By Severity — Prop. Damage	By Severity — Personal Injury	By Severity — Fatal	By Time of Occurrence — Night	By Time of Occurrence — Day	Total	Per Cent
Intersectional Friction	Obstructed view	..	8	$ 173	3	3	..	1	5	6	
	Icy pavement	..	7	1,853	12	5	..	6	11	17	
	Wet pavement	..	5	708	9	3	..	4	8	12	
	Both cars going straight through	11	305	56,620	152	158	9	76	243	319	
	At least one car making right turn	..	40	9,838	62	28	..	24	66	90	
	At least one car making left turn	13	291	55,597	296	167	10	186	287	473	
	Rear-end collision	2	91	14,490	78	55	2	62	73	135	
	Mechanical failures, tires, brakes, etc.	2	5	1,475	2	2	1	2	3	5	
	Sleepy, drunk, or careless driver	..	2	545	2	1	..	3	...	3	
	Avoiding pedestrian, animal, or vehicle	..	7	1,795	9	4	..	5	8	13	
	Others	..	...	429	3	...	..	2	1	3	
	Total	28	761	$143,523	628	426	22	371	705	1,076	18.9
Medial Friction	Marginal friction by leaving the pavement	7	69	12,894	25	35	3	31	32	63	
	Over center line, because of curves or alinement	16	225	50,433	152	111	12	180	95	275	
	Narrow bridge	..	29	8,752	45	18	..	24	39	63	
	Crossing into private or semi-public place	..	70	11,686	38	38	..	17	59	76	
	Crossing center line to pass	29	343	77,892	161	148	23	152	180	332	
	Short vertical sight distance	..	14	3,549	10	9	..	7	12	19	
	Icy pavement	6	154	35,445	101	70	5	81	95	176	
	Wet pavement	9	114	21,161	74	48	7	81	48	129	
	Sleepy, drunk, or careless driver	4	135	43,125	66	68	2	98	38	136	
	Mechanical defect, tires, brakes, etc.	5	48	10,669	28	14	4	20	26	46	
	To avoid animal, pedestrian, or vehicle	7	61	25,602	32	23	7	26	36	62	
	Others	25	201	46,511	152	104	13	202	67	269	
	Total	108	1,463	$347,719	884	686	76	919	727	1,646	28.9
Internal Stream Friction	Narrow roadway	..	..	....	..	..	..	..	..	..	
	Narrow bridge	..	1	812	10	1	..	1	10	11	
	Fixed objects in traveled way	..	2	737	9	1	..	6	4	10	
	Leaving lane to right, no road intersection	..	11	2,164	29	9	..	8	30	38	
	Leaving lane to left, no road intersection	1	84	17,507	108	50	1	57	102	159	
	Entering lane from side, no road intersection	2	73	18,036	82	39	2	47	76	123	
	Parked, slow, or stopped car on road	24	596	130,745	547	374	21	542	400	942	
	Passing other car	4	187	34,876	155	98	4	104	153	257	
	Mechanical defect, tire blow, etc.	..	3	1,819	6	3	..	6	3	9	
	To avoid animal, pedestrian, or vehicle	..	14	3,200	14	9	..	7	16	23	
	Sleepy, drunk, or careless driver	1	25	4,408	10	16	1	22	5	27	
	Others	5	15	1,792	11	12	4	18	9	27	
	Total	37	1,011	$216,096	981	612	33	818	808	1,626	28.6
Marginal Friction	Narrow bridge or culvert	..	7	2,465	8	5	..	8	5	13	
	Poor shoulders	2	45	9,579	34	25	1	32	28	60	
	Curve or alinement	16	102	25,057	67	55	13	88	47	135	
	Icy pavement	6	56	20,969	105	47	5	62	95	157	
	Wet pavement	7	114	21,806	91	69	6	73	93	166	
	Fixed objects—ditches, poles, trees, etc.	3	19	3,069	17	11	3	15	16	31	
	Mechanical defect, tire blow, etc.	5	118	20,341	56	62	5	57	66	123	
	To avoid animal, pedestrian, or vehicle	2	181	42,336	154	130	2	146	140	286	
	Sleepy, drunk, or careless driver	8	67	15,575	38	42	8	63	25	88	
	Others	4	58	12,597	58	40	4	65	37	102	
	Total	53	767	$173,794	628	486	47	609	552	1,161	20.4
Pedestrian	Pedestrian crossing pavement at intersection	1	8	627	...	8	1	6	3	9	
	Pedestrian crossing between intersections or standing on pavement	17	61	451	...	59	17	27	49	76	
	Pedestrian walking along pavement	17	37	471	...	35	17	41	11	52	
	Others	..	2	310	...	2	..	1	1	2	
	Total	35	108	1,859	...	104	35	75	64	139	2.4
Others	Railroad crossings	12	20	5,335	8	10	7	10	15	25	0.4
	Mechanical defects not classified under one of the friction groups	2	8	16,077	13	6	1	12	8	20	0.4
	Grand total	275	4,138	$904,403	3,142	2,330	221	2,814	2,879	5,693	100.0

TABLE II. ACCIDENTS TABULATED WITH RESPECT TO TRAFFIC VOLUME AND NUMBER OF LANES

	2-LANE ROADS							3-LANE ROADS	4-LANE UNDIVIDED	4-LANE DIVIDED
	Less than 500	500 to 1,000	1,000 to 2,000	2,000 to 3,000	3,000 to 4,000	Over 4,000	All 2-lane			
Length in miles	410.43	650.63	1,358	667.93	211.75	41.17	3,339.91	71.63	92.71	21.36
Average daily traffic volume	306	744	1,460	2,400	3,400	4,790	1,530	3,890	5,460	2,380
Accidents:										
Intersectional	28	77	328	268	127	30	858	61	138	9
Medial	36	138	472	535	250	47	1,478	78	79	0
Internal stream	22	112	494	468	185	56	1,337	109	173	14
Marginal	21	122	394	326	152	30	1,045	33	80	9
Total	107	449	1,688	1,597	714	163	4,718	281	470	32
Accident rate per million vehicle miles:										
Intersectional	0.613	0.436	0.452	0.459	0.484	0.416	0.460	0.600	0.746	0.485
Medial	0.788	0.781	0.653	0.915	0.950	0.652	0.793	0.765	0.429	0
Internal stream	0.481	0.634	0.683	0.802	0.704	0.779	0.718	1.070	0.936	0.754
Marginal	0.460	0.690	0.543	0.558	0.579	0.417	0.561	0.324	0.433	0.485
Total	2.342	2.541	2.331	2.734	2.717	2.264	2.532	2.759	2.544	1.724

Intersection accidents, it would appear, should vary with the units of exposure, and one would normally think that the exposure units would not be proportional to the vehicle-miles of traffic. But apparently, from the data in Table II on intersection accidents on 2-lane pavements, the exposure units and intersection accidents must vary with some uniformity with the vehicle-miles of traffic. This is particularly true when the mileage is large. The greatest variation is on roads carrying traffic less than 500 vehicles per day. Data have not been collected to prove the point, but it would seem that where volumes are less than 500 per day the operators do not consider the state road as preferential, and enter at intersections without proper precautions. The desirability of further studies on this is indicated.

Table II indicates clearly that the intersection accident rate per million vehicle-miles goes up as the number of lanes of undivided pavement is increased. The rate for 4-lane divided pavement is very close to that for 2-lane pavements.

With pavement and traffic conditions the same, one would expect marginal accidents to vary uniformly with total vehicle-miles of traffic. It is hard to believe that the variations shown for traffic volumes below 500 per day and for volumes between 500 and 1,000 per day are caused by traffic condition or driver attitude. Therefore there must be some road condition which is yet to be determined causing these variations from the average rate. The slight drop in both intersectional and marginal rates for daily volumes above 3,500 may be due to better road conditions because of better marking or maintenance, or it may be due to increased vigilance of operators where larger traffic volumes exist. The average marginal accident rate for 3- and 4-lane pavements within the volume range studied is better than for 2-lane pavements.

Internal stream accident rates show a gradual increase per million vehicle-miles from the low daily volumes to between 2,000 and 3,000, where the peak rate is reached for 2-lane pavements. The drop for volumes between 3,000 and 4,000, and the slight rise between 4,000 and 5,000 volumes, it seems may be caused largely by operator's reactions to these particular traffic densities. The following is only a thought, and cannot at present be substantiated by factual data: When the traffic volume reaches 3,000 to 4,000 per day, operators, to avoid internal stream friction, try to pass the object ahead by either the center-line route or the edge route, which tends to reduce the internal stream friction and increase the marginal and medial frictions. Both show an increase with this traffic volume range. When the volume reaches 4,000 or over, the meeting vehicles are in such number that passing is considerably restricted, speed is reduced, the operator becomes more alert, and the combined accident rate is lowered.

The internal stream accident rate for 3- and 4-lane undivided pavements within the volume range studied is higher than for 2-lane pavements. Further, the rate for 3-lane pavements if higher than for 4-lane, which is in accordance with the popular belief concerning the dangers of 3-lane roads.

Medial friction accident rates might be expected to increase rapidly with the increase in daily volume, other factors being the same, and this seems to be true within the volume groups 1,000–2,000; 2,000–3,000; and 3,000–4,000. When the traffic volume is less than 1,000 per day, the rate is close to the average for all 2-lane roads and is higher than might be expected. It would seem that the reason for this may be due to a road condition which prevails on such routes. Further facts and study are necessary to determine this. There is a decided drop in the medial rate when the volume is over 4,000 per day. It would seem that this may be due to considerably fewer attempts to pass and use the opposing lane for any purpose because of the larger number of meeting vehicles.

In respect to average medial friction accident rates very little difference is indicated between the 2- and 3-lane pavements, but there is a decidedly better rate for 4-lane pavements. The 4-lane divided pavements will have substantially no medial accidents if the division strip is 20 ft wide or more.

The last line of Table II gives total accident rates per million vehicle-miles for the various traffic densities and number of lanes. For traffic volumes below 2,000 per day the rate is near or less than the average for all 2-lane roads; for volumes of 2,000 to 4,000, the rate is above the average; and for volumes of over 4,000 it is again below the average. This variation reflects the characteristics of the separate classifications, and additional data are desirable. The average rates show the relative hazard or safety of 2-, 3-, and 4-lane pavements and of the various groups of 2-lane pavements studied.

SIMILAR STUDIES OF CONVENIENCE AND ECONOMY NEEDED

These ratios or units of measurement of safety and methods of measurement are no doubt crude, but the writer feels that they represent progress in the right direction, and can be corrected and refined as additional facts are gathered. Similar work on measuring convenience and economy—the other two traffic characteristics—should be made and superimposed on this to get a complete picture of the problem. Thus the proper balance between the three characteristics could be determined, and the traffic engineer would be in a position to provide motor vehicle transportation with the greatest possible safety, convenience, and economy.

The Corpus Christi Sea Wall Project

12,000 Ft of Concrete Structure to Protect Texas Port Against Repetition of 1919 Hurricane Disaster

From a Paper Recently Presented Before the Texas Section

By E. N. Noyes, M. Am. Soc. C.E.

Consulting Civil Engineer, Myers and Noyes, Dallas, Tex.

PROTECTION against flood from hurricane is the primary purpose of the sea wall project now under construction in Corpus Christi, Tex. Its completion, however, will provide the city with several other benefits as well—including a two-mile length of shore-line boulevard, and three boat basins. Financing, design, and progress to date are all discussed by Mr. Noyes in the present article.

LIKE all other cities along the Gulf coast, the port of Corpus Christi, Tex., is subject to tropical hurricanes. Originating in the South Atlantic or the Caribbean Sea, these storms trace a curve to the northwest across the Gulf and then swing into land in a northerly and northeasterly direction, causing destruction by wind, wave, and tide.

In 1919 one of these hurricanes struck Corpus Christi, with a serious loss of life and property. The general elevation of the lower section of the city varies from 5 to 9 ft above mean tide. The tide and waves built up by the storm, reaching an elevation of +11.5, swept through the entire low-lying section, and with the accompanying drifting timbers and debris battered everything in their path. The town was entirely isolated so far as rail, highway, and wire communication was concerned.

This disaster led to the passage of legislation, in 1921, whereby a portion of the state ad valorem taxes on seven Gulf coast counties was remitted for a period of 25 years, beginning September 1, 1921, for the purpose of "aiding the city of Corpus Christi in the construction of sea walls and breakwaters so as to protect said city from calamitous overflows."

With a bond issue based on this tax remission, a loose stone breakwater was built in front of the city in 1924 (Fig. 1). This structure was not of the type or height desired, but was all that could be done with the funds then available.

By 1938, additional tax remission funds to the amount of approximately $900,000 had accumulated, as the result of an increase in taxable property values by virtue of discovery of numerous oil fields and the increase in agriculture and cattle raising, and it was decided to construct more adequate protective works. The new plans called for a sea wall that would completely enclose the city south of the Ship Channel, against waves and tides to an elevation 2.5 ft. above those of the 1919 storm. The estimated cost of the work was $1,600,000. A PWA grant-in-aid was sought, but was not obtained. The city decided, however, to proceed with the work to the extent of the funds available, and seek elsewhere for sufficient money to complete the project. In June 1939 the legislature extended tax remission for ten years, to 1956, estimated to give $1,500,000 of additional funds with which to complete the project.

The selection of a design was determined by the following factors:

1. Safety above the greatest storm of record, that of 1919
2. Drainage of storm water off the protected lands
3. Control of waterfront by the city
4. Improvement of the appearance of the waterfront
5. Reclamation of private lands in exchange for release of riparian rights
6. Provision for yacht basins
7. Prevention of backwater coming in from the turning basin and the Industrial Canal
8. Future widening of Water Street

Adoption of general plans followed an inspection of similar work along the Gulf coast and on the Atlantic coast from Key West to Atlantic City. The principal feature of the project is a concrete structure of the step type, supported on piling, backed up by a hydraulic fill, and incorporating a steel sheet-pile cutoff wall (Fig. 2). This step structure will extend from near the Bascule Bridge on the north (Fig. 1) to the bluffs south of the city, a distance of approximately 12,000 ft.

At the north end of the project, the Port Commission is to pay the cost of 400 ft of wall along its property line; while across the Government property used by the U. S. Engineers, the Government will construct 900 ft of wall.

A Completed Section of the New Sea Wall

This View Was Taken from the Municipal Wharf, Looking North. The Concrete Culvert in the Foreground Passes Through One of the Yacht Piers and Is Designed to Create a Circulation of Water in the Yacht Basin

At Work on the Sea Wall

At the Right the Graded Hydraulic Fill Can Be Seen. Near the Center Is the Traveler on Which the Beam Steel Is Formed and Lowered Into Place. Beyond, to the Left, Is the Traveling Form, Where Concrete Is Poured

making a total of about 1,300 ft of wall that will be paid for by parties other than the city.

In order to prevent backwater from entering the city through low-lying ground west of the Turning Basin warehouses, a backwater levee is to be constructed from the high ground on the north edge of the city, near the Missouri Pacific Station, north to the fill along the Turning Basin. This levee will cross a number of tracks of the Southern Pacific Railway, and to accommodate these tracks without the necessity of their being raised, a gate structure which can be closed at times of high water has been designed.

CLOSE-UP OF REINFORCING STEEL IN PLACE

The Chains Under the Reinforcing Are Weighting Down the Heavy Kraft Paper Which Separates the Fill from the Concrete

In addition to the sea wall proper, plans call for a certain amount of construction to give a safe harbor for small boats. The first three boat basins will have an area of approximately 60 acres with a minimum depth of 8 ft, the areas being enclosed by T-head piers. The T-heads will be 300 ft by 1,000 ft, connected to the wall by a pier 65 ft wide carrying a roadway over the wall on a ramp. They will be developed for recreation purposes. Additional safe berthing space for small boats can be added as the demand arises.

The location of the sea wall is approximately 500 ft east of Water Street, and the hydraulic fill will occupy the entire intervening space, rising on a grade of approximately 2 per cent to the west line of the proposed Shoreline Boulevard. This boulevard will include a 20-ft splash walk adjacent to the wall; two roadways, each 40 ft wide; an intermediate park area varying in width from 80 to 300 ft; and a parking and sidewalk strip on the west, with a minimum width of 15 ft.

Property under private ownership extends east from Water Street (the present approximate water line of the bay) a distance of 300 ft, the greater part of this land being under water. This property carried with it certain riparian rights which were considered to have value, but of an undetermined amount. The city arranged with the owners to release these riparian rights and to pay for the filling-in of their properties in exchange for the construction of the sea wall and boulevard adjoining them on the east. As a result, the city now has complete control of its entire waterfront and of a strip back of the sea wall, varying in width from 200 to 350 ft and extending from the south line of the Government property to the bluffs on the south, a distance of approximately 11,000 ft. It will be noted that this control covers the entire bay front opposite the business area of the city.

In the design of the sea wall structure, an important consideration was the live load that might be expected to be produced by the waves. The wave height at the existing breakwater is computed to be about 6.2 ft, using the Stevenson formula, $H = C\sqrt{F}$, with $C = 1.61$ and $F = 15$ miles—the distance to the chain of islands lying offshore. Because of the partial protection afforded by the breakwater, wave height at the sea wall can be expected to be somewhat less than 6.2 ft, and a height of 5 ft was assumed. To compute the wave pressure, the Gaillard formula and others were applied. The pressure arrived at on the basis of this formula is 800 lb per sq ft, of which half is the result of the potential energy and half the result of the kinetic energy, of the wave. On a typical step the pressure due to potential energy, normal to the wall surface, is 2.02×400, or 808 lb. The pressure due to kinetic energy acts in a horizontal direction against the face of the step and, resolved to a line normal to the slab, equals 198.5 lb, making a total of 1,006.5 lb. This value, divided by 2.02, gives a live load of 498 lb per sq ft of slab. To this is added a dead load of 187 lb, making a design total assumed at 700 lb per sq ft.

The working stress in the concrete is assumed at 750 lb per sq in., and the specifications are designed to give

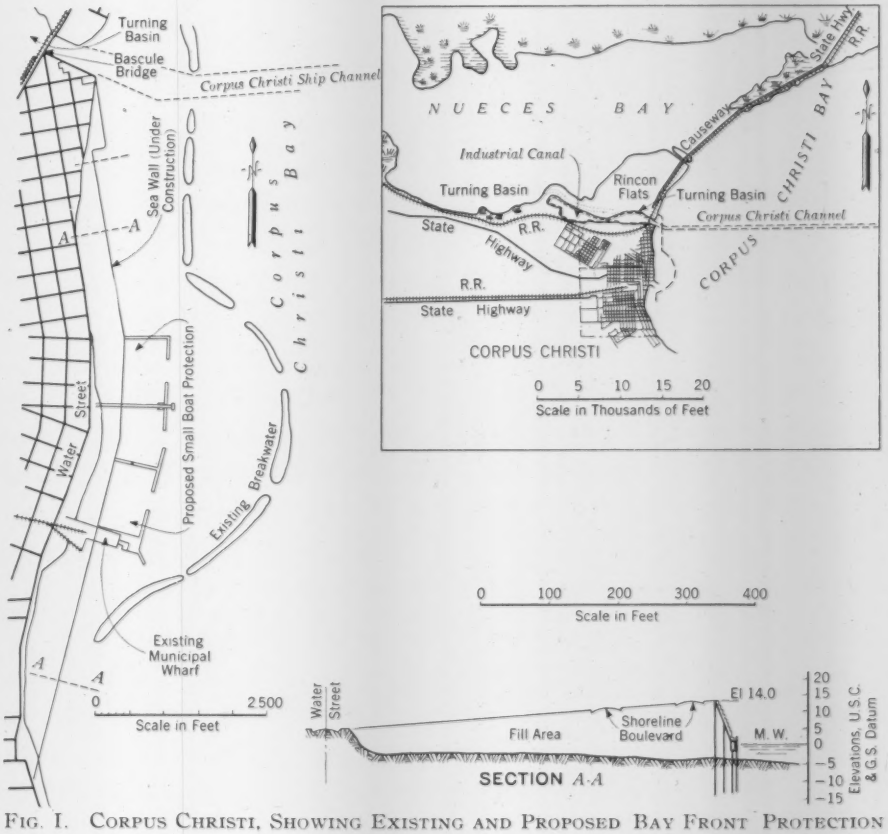

FIG. 1. CORPUS CHRISTI, SHOWING EXISTING AND PROPOSED BAY FRONT PROTECTION

a concrete strength of 3,500 lb per sq in. Stress in reinforcing steel is 18,000 lb.

Borings were taken at 300-ft intervals along the line of the wall, and a length of steel piling was driven for tests about every 600 ft. On the basis of these borings and pile tests, the length of steel sheet piling adopted varies from 15 to 20 ft at the south end, up to 34 ft in the vicinity of the Barge Docks. These lengths carry the cutoff wall into good material at all points.

Bearing piles for the wall were originally designed to be of either pre-cast concrete or 20-lb creosoted pine, a considerable investigation having led the engineers to the conclusion that the latter type, particularly where protected from the action of water or air, would have a life more than sufficient to justify their use in this project. When bids were received it was found that a considerable saving could be made if creosoted piling were adopted, and accordingly this type was chosen.

CURING IN PROGRESS, AND A SECTION OF FINISHED WALL
The Fill in the Foreground Is for the Construction Road. It Will Later Be Removed and Used for Additional Fill Behind the Wall. Mean Tide Elevation Is at About the Mid Height of the Bottom Step

The dead load on each bearing pile, disregarding the considerable value of the steel sheet piling and the supporting effect of the hydraulic fill, is 18 tons. The piles vary in length from 29 to 59 ft, depending upon driving conditions. Bearing values were determined from the Engineering News Formula, and do not take into consideration any additional value that may result from skin friction in the hydraulic fill.

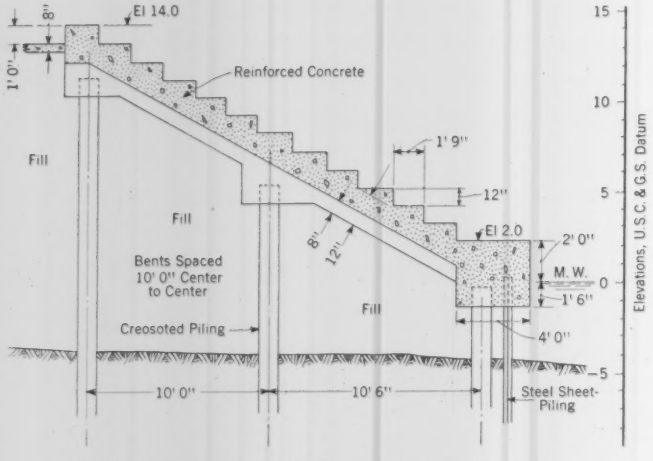

FIG. 2. TYPICAL SECTION OF CONCRETE WALL

Specifications for construction of the sea wall call for the concrete toe, which extends $1\frac{1}{2}$ ft below mean tide level, to be cast in the dry. The method adopted by the contractor is to fill to an elevation of +3 or +4 over the entire area and from 50 to 60 ft outside the line of the wall; and then excavate a trench in which to drive the sheet piling and pour concrete. The trench is unwatered by means of pumps. After the sheet piling is driven, the hydraulic fill is carried up to grade along the line of the wall. It is then surfaced, and covered with a double Kraft paper on which the concrete for the wall is poured.

Principal cross-sectional dimensions of the wall are shown in Fig. 2. Bearing piles are on 10-ft centers in each direction, and expansion joints are placed at 40-ft intervals, using a keyway filled with expansion-joint material. All reinforcing is buried at least 3 in. clear, from the face of the concrete.

One of the most difficult problems of the entire project is the provision for taking care of storm water. At the south edge of the business section, a deep arroyo empties into the bay from a fairly large drainage area. The plans to handle this call for a 9-ft horseshoe structure, extending upstream to an elevation of about +20 and acting as a closed pressure sewer from that point to the discharge end at the sea wall.

Throughout its length, at each street intersection, gravity storm sewers extend eastward through the sea wall. All these sewers are provided with two automatic gates—one at the point of discharge and one in the manhole in the west roadway of the Shoreline Boulevard. These gates will prevent a back flow at times of high tide. Should a heavy rainfall occur at such a time, water will flow on the streets to the north and west to sluice gates through the back levee.

In selecting drainage gates, careful study was given as to the kind of metal or treatment to be used, and considerable correspondence was had with engineers who have used gates in contact with sea water. It was found that no particular value can be given to bronze seats or to galvanizing. Where a combination of metals is used, the occurrence of electrolytic action may cause serious deterioration at the point of contact of metals of different composition. Gates were accordingly specified to be of all cast-steel or cast-iron designed for a head of 22 ft.

The project as a whole includes the hydraulic fill, storm sewers, sea wall, and paving of the east roadway of Shoreline Boulevard (together with the necessary curb, gutter, walk, and splash area), from the Bascule Bridge to the bluffs on the south. In addition to this, three basins for the protection of small boats are provided. With the funds at first available, it was necessary to eliminate certain of these features—namely, the sea wall from the Municipal Wharf south, one small boat basin, the paving of the east drive, and the construction of the backwater levee. The construction contract provides that these items be reinstated in a definite order as additional funds may be secured, and these items will now be completed under the extension of tax remission.

Up to August 4, 1939, some 3,200 lin ft of wall had been poured, and the work at that time was proceeding at the rate of 40 lin ft (80 cu yd) per day. J. DePuy, of San Antonio, is the general contractor. J. C. Bisset, M. Am. Soc. C.E., is city engineer of Corpus Christi, and the writer's firm is consultant on the project.

Aggregate Selection and Concrete Mix Design

With Comments on Vibration Compaction and Water Curing

SECOND OF TWO ARTICLES ON THE ELEMENTS OF MODERN CONCRETE

By CHARLES E. WUERPEL

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THIS is the second of two articles by Mr. Wuerpel on the elements of modern concrete, designed especially for the engineer who has not been able to devote his time in recent years to the minutiae of the subject. The first article, appearing in the August issue, discussed the choice of water-cement ratio and the types and properties of cement. Now Mr. Wuerpel takes up the question of choice of aggregates, which he suggests does not always get the attention it deserves. Even so small a factor as the shape of the coarse aggregate, he points out, may have a profound effect upon the characteristics and economy of the concrete mixture. As for mix design, the "actual trial method" is recommended. There follow several comments on the selection of consistency and plasticity, and the article concludes with discussions of the uses and abuses of internal vibration and the principles of curing.

ALL too frequently, the choice of aggregate for a concrete structure is based largely upon accessibility and relative first-cost economy, the philosophy being that suitable fine aggregate is any local sand and that suitable coarse aggregate is any available crushed stone or gravel. This regrettable state of affairs arises from the mistaken conception that aggregate is solely something with which to stuff cement paste for economy, or worse yet, for added compressive strength.

To realize the best results from a concrete structure, each of the three ingredients (water, cement, and aggregate) must be considered equally important. A low and well-controlled water-cement ratio and a well-balanced cement type are of little benefit if the aggregate is unsuited for the member in which it is placed.

Importance of the choice and judicious use of aggregate may be realized by observing the profound effect which the shape alone of coarse aggregate has upon the characteristics and economy of a concrete mixture. Figure 1 contains a group of curves showing the "relative remoldability" of concrete mixtures containing materials which are identical except for the type and shape of the coarse aggregate. Before discussing this illustration, the term "relative remoldability" as used herein is defined as the time required to remold, by external vibration, a truncated cone of concrete to a flat disk in a special container called the remolding apparatus. The apparatus and idea originated with T. C. Powers, of the Portland Cement Association ("Studies of Workability of Concrete," *Journal*, American Concrete Institute, February 1932), and has been developed and given practical application at the Central Concrete Laboratory of the U. S. Engineer Department. The objective is to roughly simulate, under closely controlled conditions, the remolding of a heap of concrete, which has been dumped into a form, to its effectively consolidated final position.

Remembering that all ingredients are identical (in type, size, and grading) except for the shape and type of the coarse aggregate, consider the curve which begins farthest to the left of Fig. 1. This mixture contains a well-rounded siliceous beach gravel as coarse aggregate. The first mixture attempted contained 22 per cent (of the total aggregate present in the mix, by weight) sand and required 60 seconds to remold. An increase of 2 per cent in the sand ratio decreased the time required to remold from 60 to 30 seconds. Further increases in the sand ratio, in small increments, finally reduced the remolding time to 16 seconds at 30 per cent sand. Thus the time required to remold these ingredients, in a plastic mixture, was reduced to one-fourth by increasing the sand ratio by 8 per cent. Observe, also, that increasing the sand beyond the 30 per cent ratio caused a gradual upturn in the curve, indicating loss in placing efficiency. In this case, the 30 per cent ratio is the optimum.

Now, consider the curve at the right. This combination of materials contained angular crushed granite having a large percentage of flat and elongated particles. Observe that the first mixture attempted contained a sand ratio of 38 per cent and required a remolding time of 55 seconds. As the sand ratio was increased the remoldability improved until the optimum was reached at 47 per cent sand. Increasing the sand beyond this value brought about a lessening of the remolding efficiency.

Certainly it becomes apparent that arbitrary mixtures, such as 1-2-4, 1-2½-5, or 1-3-6, for these two types of material could not be used interchangeably. The optimum mixture with the rounded gravel was 1-2.45-5.76, whereas the crushed stone mixture was 1-2.60-2.90. The gravel mixture required 4.55 sacks of cement per cu yd whereas the stone mixture required 6.15 sacks per cu yd. Suppose a record was available of an efficient mixture design made for some project in which the rounded gravel had been used. The data appear to meet the requirements of a new project—the water ratio is right; the cement is similar; the sand and gravel size and grading meet the present specification; and the degree of workability is just the amount desired. But on the new project, crushed granite has been purchased for coarse aggregate. If the old mix design were adopted, the first batch placed would cause considerable embar-

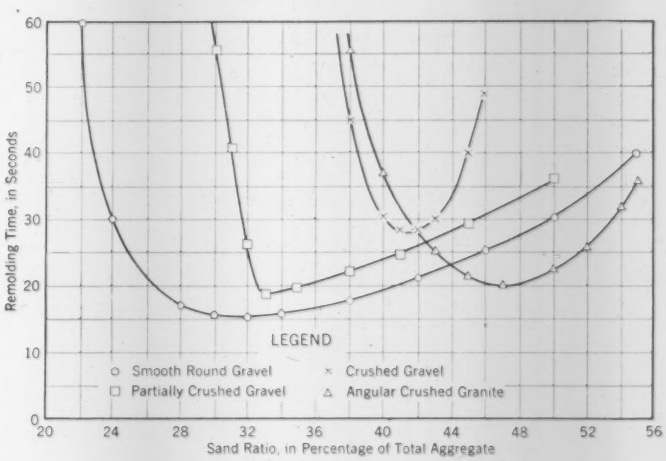
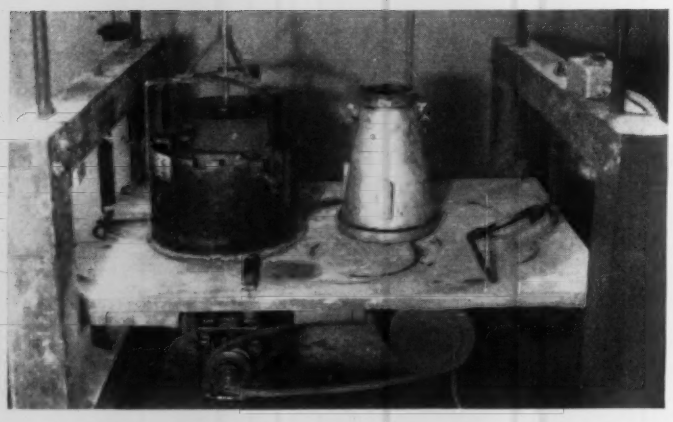


FIG. 1. REMOLDING CURVES, SHOWING EFFECT OF SHAPE OF COARSE AGGREGATE ON MIXTURE PROPORTIONS



REMOLDING APPARATUS IN THE CENTRAL CONCRETE LABORATORY
Slump Cone in Which Sample Was First Molded Has Been Removed. Remolding Is Done by Vibration

rassment. It would be so dry and harsh that it probably could not be consolidated.

The difference in shape need not be so striking as in the example just given. In fact, the ultimate dissatisfaction might be greater if the difference were much less, or reversed, because then the arbitrary mixture might be used throughout the project, with the result that there would be honeycombing, rock pockets, and segregation, or that the cement requirements would be wastefully excessive, or the labor costs unnecessarily high.

Good grading of fine and coarse aggregate is also of prime importance, because it affords a minimum of voids to be wastefully filled with cement paste and a minimum surface area to be coated and lubricated by the paste. Perfect grading is rarely obtainable and should not be sought for as much as a satisfactory grading that can be delivered to the mixer from hour to hour and day to day with a minimum of variation. A satisfactory mixture can be designed for most gradings of aggregate, but—and this should be emphasized—no design can be depended upon to absorb constant variations in grading. Constancy of grading can be obtained, but producers must be trained to produce it by continual insistence, and more than the usual cursory attention must be given the stock-piling operation to minimize or prevent the destruction of uniform grading by segregation.

The grading of fine aggregate is important in its effect upon the finished structure as well as upon the true economy of the mixture, because it largely influences segregation. The limits of permissible variation should be narrower than they usually are, and in no case should the sand contain less than 10 per cent passing the No. 50 sieve or less than 1 per cent passing the No. 100 sieve. The following is a good typical sand grading:

SIEVE	PER CENT PASSING	TOLERANCE, PER CENT
4	97	3
16	65	10
50	15	5
100	4	2

Segregation in concrete has a broader significance than that usually attributed to it; the separation from the mortar and collection of particles of coarse aggregate is only one manifestation. Of greater importance is the segregation of water (called also "water gain," "water rise," "bleeding," and "sand streaking").

The segregation of water is caused by insufficient surface area presented by the cement, sand, and coarse aggregate to hold it by surface tension. Much can be done to minimize water segregation by controlling the type and surface area of cement, but improperly graded sand is often the principal contributory cause of this undesirable phenomenon. The tangible evidences of bleeding—that is, sand streaking and water rising to the top of newly placed concrete—are not necessarily the most serious effects. As segregated water is displaced upward and outward by the greater specific gravity of the solid materials, flow channels and capillaries of large size are formed, and, since particles of aggregate and horizontal steel reinforcing bars restrict the upward movement, water is trapped on the under sides of these obstacles with a resulting complete prevention of bond. A vertical column of concrete broken horizontally will disclose that the irregular cleavage follows the under sides of coarse aggregate particles. If water pressure is applied in a direction normal to the direction in which the concrete is placed, flow is most likely to develop along paths formed by the under sides of aggregate particles. This condition becomes more serious as the tendency toward water segregation increases.

Fine and coarse aggregate must be structurally sound and inherently resistant to climatic or chemical attack if it is to be used in a structure which will be exposed to frost action or aggressive waters. The requirement for soundness is one of the most rigid of the acceptance requirements for aggregate used in the flood control work of the U. S. Engineer Department in the northeastern United States. It is rather simple to detect unsoundness in ledge rock, but glacial and alluvial gravels present a more difficult problem. Close visual examination of a gravel bed or stock pile by even a competent technician is not sufficient for a quantitative analysis of soundness. The gravel must be tested by an actual freezing and thawing test or by a crystallization test. The Central Concrete Laboratory uses both, but places greatest reliance upon the magnesium sulfate (crystallization) accelerated soundness test. In the latter, carefully separated particle sizes of the fine and coarse aggregate are subjected to 10 cycles of saturation (in a concentrated solution of magnesium sulfate) and dehydration at 105 C. If the material loses more than 10 per cent by weight it is rejected unless a positively verified service record indicates that it is suitable for the proposed use.

EFFECTS OF AGGREGATE ON PROPERTIES OF CONCRETE

Aggregate affects the structural qualities of hardened concrete in a number of ways. For example:

1. The modulus of elasticity of concrete, similar in every respect except coarse aggregate, may vary from 2,000,000 to 6,000,000 lb per sq in., the lower value being obtained with a soft limestone aggregate and the higher with a crushed diabase rock.

2. The coefficient of thermal expansion of concrete may vary from 0.000004 to 0.000006 in. per in. per deg F, depending upon the coarse aggregate.

3. The flexural strength of concrete is materially affected by the type and shape of the coarse aggregate; the compressive strength is affected but slightly.

4. The specific heat, thermal conductivity, and coefficient of heat diffusion are all affected by the type of coarse aggregate.

5. The plastic flow, the moisture coefficient of expansion, and the shrinkage coincident with cement hydration are affected by the aggregate, both fine and coarse.

From the foregoing, it may be seen that a structure may be benefited or hurt materially by the choice of aggregate.

The article in the August issue, and the foregoing part of the present article, have been devoted to the component materials of concrete, treated as more or less

separate entities. The problem of combining these materials effectively is next to be considered.

There are numerous methods of arriving at the proportions for a concrete mixture. Of them all, the writer prefers the actual trial method. The apparatus required is not necessarily extensive and may consist of a shallow metal pan, or a table, or a wheelbarrow; a blunt-ended mason's trowel or two; a small set of scales; several buckets; a puddling rod; and a slump cone.

A fraction of a bag of the cement purchased for the job is weighed out and placed in the mixing pan. The proper amount of water, based upon the chosen water-cement ratio, is added to the cement and the two are thoroughly mixed into a paste. Sand and gravel are then added in gradual increments from previously weighed amounts (the surface moisture content having been determined, or the aggregate being in a saturated, surface-dry condition) until, with continuous mixing, the mixture assumes the approximate consistency and plasticity desired. A slump test is made and the result recorded. Then by deducting the weights of sand and gravel remaining from the originally weighed amounts, the quantities which have been added to the paste are accurately determined. With the specific gravity of the separate materials known, it is then possible to calculate the absolute volume of concrete which has been made with that particular fraction of a sack of cement. From these data the amount that can be made with a full sack of cement, and the weight of each material required to make 1 cu yd of concrete, are computed.

The first time this method is attempted there is every possibility that the mixture will get out of hand, due to the excessive addition of either sand or gravel; but with a little practice it is surprising how close to the optimum mixture the first trial will come. Subsequent batches should be made with predetermined amounts of material based upon the data calculated from the pure trial batch. A good method is to compute the sand ratio obtained in the original batch and calculate several additional batches with sand ratios somewhat greater and somewhat smaller than the original. The optimum batch can then be determined as the one which gives the greatest appearance of plasticity at the desired slump with the minimum cement content. With the use of the remolding machine, judgment as to the maximum plasticity, optimum sand ratio and cement factor is materially aided.

In selecting the proper consistency and plasticity (remoldability) for a concrete mixture, the controlling factors are (1) the means of transporting the concrete from the mixer to the forms, (2) the means of placing the concrete in the forms, (3) the means of consolidating the concrete into its final position, (4) the congestion of the form interior due to reinforcing steel or embedded items, and (5) the shape of the monolith being placed.

The principal means of conveyance are transit mixer, bottom-dump bucket, chute, belt, pump, compressed air, and the time-honored buggy. Each of these requires particular degrees of plasticity to obtain maximum efficiency and economy.

The principal aids to consolidation within the form are internal vibration, external vibration, spading and forking, and booting. Each of these, also, requires a particular degree of plasticity for maximum efficiency and economy, but in this respect internal vibration is the most critical, and when it is to be used the concrete mixture must be very carefully designed to give proper results. The sand ratio must be lower than for other methods and the optimum ratio must be carefully determined. Internal vibration sets up radiating oscillations which cause the materials in the mixture to settle into the densest possible mass. Any excess of material beyond that required for maximum density, be it cement, sand, stone, or water, will be excluded from this dense mass if vibration is prolonged.

SEVERE FROST ATTACK ON PRE-CAST CONCRETE BLOCKS, DUE TO POROSITY AND UNSOUND AGGREGATE

USES AND ABUSES OF INTERNAL VIBRATION

Internal vibration is one of the major advances in the concrete industry, but it must be used intelligently. When so used, economy, heat reduction, increased volume stability, and resistance to chemical attack are effected by an appreciable reduction in the amount of cement necessary (as much as 1 or $1^1/_2$ sacks per cu yd); greater and more uniform density may be achieved; and increased strength is possible through the practicability of using a lower water-cement ratio as an alternative to a reduction in cement content.

Unfortunately, internal vibration is often abused. Frequently the attempt is made to consolidate concrete, designed for vibration, with insufficient or inefficient vibrators. Again, internal vibration is often mistakenly applied in steel-congested and narrow areas which require a fairly fluid mixture to flow around the interior obstacles. With such a mixture, vibration will result in serious segregation. If, on the other hand, the mixture is designed to prevent segregation with vibration, it will be too dry to place properly in the forms.

There is considerable argument about the optimum frequency of vibration. This question is far from being settled, but the writer feels safe in saying that satisfactory consolidation may be achieved with a vibration speed of not less than 3,600 rpm—this speed to be measured when the vibrator is immersed to its full operating depth in the concrete to be placed. It should be noted that there are vibrators on the market whose manufacturers' claim for vibration speed is based upon their operation in a free condition. These vibrators may show as much as 5,000 rpm before immersion in concrete, and drop below the minimum desirable (3,600 rpm) when fully immersed. This indicates the desirability of testing each vibrator purchased in the concrete to be placed. A Frahm tachometer may be used for the purpose.

The effectiveness of a vibrator depends largely upon the relation of its bulk and amplitude of vibration to the size of the member and the condition of the concrete being placed. The small, tubular, flexible-shaft vibrator of high frequency, low amplitude, and small bulk is particularly well suited for structural members and thin sections in which a fairly fluid mixture must be used. It is easily handled and can be made to reach almost inaccessible spots within the form. This type will handle from 5 to 10 cu yd of concrete per hour. The large, two-man, heavy-duty type, which is used in the open sections of large dams, will handle up to 40 cu yd of dry, harsh-appearing concrete per hour. Whatever type of vibrator is used, spare units should be available at the site to take care of breakdowns and permit uninterrupted placing of concrete.

The radius of effective vibra-

VIBRATING 3/4-IN.-SLUMP CONCRETE
Left: A High Cone of Concrete, Before Compacting. Right: "Pulling Down the Cone"—45 Seconds Later

tion is a function of the acceleration of the vibrator and the inherent plasticity of the mixture, and in most cases it does not exceed 2 or 3 ft. The depth of effective vibration is the depth to which the vibrator tip is allowed to penetrate. The time of effective vibration is wholly a matter of observation and experience. Theoretically, excessive vibration is harmless—on the premise that once maximum density is achieved no more can be done. Practically, a mixture is rarely designed that has no excess of any of its ingredients; therefore excessive vibration will cause segregation. A good rule is to apply the vibrator in as near to a vertical position as is practicable and continue vibration until a sheen of mortar first appears at the surface.

Permitting the internal vibrator to come into contact with the vertical reinforcing steel has been proved to be beneficial to bond rather than detrimental, providing the steel is well tied and will not be displaced. The oscillations transmitted downward below the zone of vibration are rapidly damped out, but, to that depth to which vibration does reach into quiescent concrete, an improvement in bond between steel and concrete is effected.

Permitting the vibrator to come near to or in contact with the form is beneficial in aiding the escape of entrained air, and detrimental only to the extent to which it may damage the form surface by denting or burring.

During the period of vibration, full hydrostatic pressure of about 150 lb per sq ft is exerted laterally in the immediate vicinity of, and to the effective depth of, the vibrator. If a long, flexible-shaft vibrator is used to a considerable depth in a thin, high-lift wall, the form pressure may be quite high, because the entire column of concrete above the tip of the vibrator may act as a fluid. On the other hand, concrete placed in shallow courses and vibrated at correspondingly shallow depths may exert less total form pressure than would a soupy concrete placed entirely by hand. But in any case the form joints should be tight and the forms well braced.

A curious phenomenon of internal vibration is that, although internal friction is reduced to cause full fluid pressure to act in the zone of effective vibration, when vibration is discontinued the coefficient of internal friction becomes quite high, with a very steep angle of repose.

IMPORTANCE OF PROPER CURING

The motto, "Save the surface and you save all," applies as well to concrete as it does to paint. Improper curing affects not only the appearance of concrete, but its durability, permeability, and strength as well. Apparently the reasons for the need of proper curing are often misunderstood. The presence of an excess of water on the exposed surfaces of hardening concrete is essential to the maintenance of stress equilibrium as well as to the proper hydration of cement. For illustration, consider a sphere of cement mortar whose exact center is occupied by a grain of hydrating cement. There is more water present than is necessary for complete hydration, but as the center particle absorbs water from the surrounding area, the expansion of the particle is not equal to the amount of water absorbed. To maintain equilibrium, water is drawn inward toward the center. As hydration progresses throughout the sphere, more and more water is drawn inward to replace that which is being drawn into the millions of cement particles. Ultimately the water content of the surface, if it has no reservoir to draw from, is seriously depleted; shrinkage consequently occurs, and tensile cracks develop. Add the effects of evaporation to the effects of this indrawing of water toward the center, and there is a resultant drying of the surface which not only causes cracks but actually inhibits complete hydration of the cement. This may be considered an exaggeration by some readers, but it follows the thought that we frequently build into concrete differential internal stresses which later affect materially the stability and durability of the structure.

For curing to be completely effective, an excess of water must be present constantly on all exposed surfaces. The practice of periodic wetting is not only inefficient, but actually injurious, because the rapid expansion and contraction of the concrete surfaces, with alternate wetting and drying, causes excessive volumetric stresses.

Hydraulic cement will hydrate only in the presence of an excess of water. Considering the rate of hydration of the major compounds of cement, the ideal condition is to maintain an excess of water for a period of at least six months in order to realize the maximum practicable benefit from the cement. That is, of course, impracticable of achievement under most conditions, but excess water should be maintained for the longest time possible—certainly not less than 7 days, and if possible not less than 14 days. The C_3S content of cement does not complete hydration, even in the presence of water, until about 28 days, and the C_2S content does not begin to hydrate until after about 14 days. As much as 75 per cent of the cement's efficiency may be lost if curing is stopped prematurely. Fortunately, this loss is restricted to the area near the surface. Capillary attraction greatly retards moisture loss, and dense concrete walls 2 ft or more in thickness may show no moisture loss at depths greater than 4 in. even after six months of exposure to air with a relative humidity of about 50 per cent.

The question of substitutes for water curing frequently arises. There is no substitute known to the writer that is equal to an excess of water on the surface of concrete. The various patented curing materials can, at best, only prevent evaporation, and few accomplish that effectively

Preserving the Central Valley

A Brief Sketch of the Reclamation Bureau's Vast Project in California

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SUPERVISING ENGINEER, U. S. BUREAU OF RECLAMATION, SACRAMENTO, CALIF.

CALIFORNIA geography, which is continually being revised to better suit the use and convenience of mankind, is now undergoing another major transformation. With the completion of the Central Valley Project, water that has almost reached the ocean via the Sacramento River will be pumped a hundred miles back upstream and turned into the San Joaquin; artificial rivers will stretch in two directions from Friant Dam; reclaimed delta lands will be protected from salt water intrusion; and channels long choked with silt will again have adequate water depths for commercial navigation. A quick survey of the project, with special emphasis on its present status, is Mr. Young's purpose in the accompanying paper. It has been prepared from a lecture on the Waterways Division program at the 1939 Annual Convention.

THE Central Valley Project, latest of the Federal Government's great water conservation enterprises, actually was first proposed 65 years ago, in a little-known document called the Alexander report. That document, transmitted to Congress by President Grant in 1874, envisioned a comprehensive system of irrigation for California's Great Central Valley, including some canals almost identical in location with the main canals of the project under construction today.

Lieutenant Colonel B. S. Alexander of the Corps of Engineers, chairman of a Board of Commissioners appointed in 1873 to investigate and report on a system of irrigation for the valley, recognized the fundamental water problem of the West when he wrote: "The subject of irrigation is a novel one to the inhabitants of the states lying east of the 100th meridian where the harvests are so uniformly assured that a season of five or six weeks of continuous drought during the growing of crops would be looked upon as a national calamity and prayers doubtless would be offered in the churches for rain. In the East the yearly rainfall is somewhat regularly distributed through the different months; but on the Pacific Coast there are two very marked seasons—one long, dry, and almost cloudless, embracing parts of the spring, all of the summer, and part of the autumn; the other season comparatively short and wet."

Colonel Alexander recognized also the need for geographic as well as seasonal redistribution of water in the Central Valley when he advocated a series of grand canals diverting both right and left from the main rivers in the foothills and skirting the edges of the valley to irrigate several hundred thousand acres of uplands where the natural water supply was inadequate. He likewise was interested then, as we are today, in navigation, and his report states that, "The canals for irrigation of the valley, owing to the necessity of giving them a gentle slope, may easily be made navigable."

Since Colonel Alexander's time the Great Central Valley, comprising the basins of the Sacramento and San Joaquin rivers, has advanced from a vast expanse of grain and cattle ranches to an empire of diversified agriculture, populated by a million people and representing a two-billion-dollar investment which is wholly dependent upon an adequate supply of that most precious natural resource—water. Yet despite the fact that about 3,000,000 acres are now under irrigation, with the normal flow of many streams fully appropriated, Central Valley basin waters still are largely uncontrolled. Agricultural development has far outstripped the unregulated water supply. To preserve that rich development and prevent threatened retrogression of parts of the area is the major purpose of the Central Valley Project. This undertaking involves regulation and redistribution of the waters of the Sacramento and San Joaquin rivers for purposes of navigation improvement, flood control, supplemental irrigation, salinity control, and electric power development.

The story of this project, as given here, is not intended to be comprehensive. Rather, by concentrating on a few features, I wish to illustrate some of the existing conditions that are in need of correction, and then tell something of the construction initiated to date, with occasional reference to work contemplated in the near future.

As will be seen in Fig. 1, the Great Central Valley is in reality two valleys—the Sacramento and the San

U. S. Bureau of Reclamation Photo

YEAR-ROUND COMMERCIAL NAVIGATION, NOW CONFINED TO THE LOWER REACHES OF THE SACRAMENTO RIVER, WILL BE GREATLY EXTENDED BY THE PROPOSED OPERATION OF SHASTA RESERVOIR

Potential Savings in Transportation Costs on Commodities Moving Between the Sacramento Valley and the San Francisco Bay Region Have Been Estimated at $2,250,000 Annually

U. S. Bureau of Reclamation Photo

GRAVEYARD OF AN ORANGE GROVE IN THE SAN JOAQUIN VALLEY
Broken Sections of Torn-Up Irrigation Pipe Bear Witness to the Water Supply That Failed. The Central Valley Project Will Furnish This Area with Supplemental Water Through the Friant-Kern Canal

Joaquin. The Sacramento River flows down from the north and the San Joaquin comes from the south, to meet in a common delta, mingle in a network of channels, and issue together through the San Francisco Bay system to the Pacific Ocean. The Central Valley is almost 500 miles long from Mount Shasta to the Tehachapi Divide, about 50 miles wide between the Sierra Nevada and the Coast Range, and varies in elevation from sea level to over 400 ft. Nature has endowed it with relatively rich soil, a semi-arid climate conducive to subtropical fruit culture, and several million acres of potential agricultural land surrounded by a mountain watershed of 40,000 sq. miles.

WATER RESOURCES OUT OF BALANCE GEOGRAPHICALLY

The fundamental reason for the Central Valley Project is that within this area the water resources are out of balance not only seasonally, but geographically as well. The Sacramento basin has tributary watersheds producing two-thirds of all the water in the valley, and crop lands with only one-third of the valley's irrigation need; whereas the San Joaquin basin has one-third of the water, and crop lands with two-thirds of the irrigation need.

Perhaps the most critical feature of the conservation problem is to be found in the southern San Joaquin Valley where at least 400,000 acres of wealth-producing lands are dangerously short of water. Almost all the normal surface flow in this region was fully appropriated for irrigation years ago, and the farmers resorted to pumping water from underground. As this type of irrigation increased, ground-water levels fell—and the farmers discovered, years too late, that their draft upon the subterranean supply was greater than its annual natural replenishment by rainfall and stream flow. What this creeping paralysis of aridity means to the valley is strikingly illustrated by an accompanying picture of an abandoned orange grove. But crops flourish when the fertile soil is adequately watered, and under proper irrigation the choicest products of both the temperate zone and the sub-tropics are grown to perfection.

In California's so-called "Little Holland," at the common delta of the Sacramento and San Joaquin rivers, the fertile peat and silt swamps have been reclaimed over a period of 75 years and placed under irrigation through a myriad of channels. However, the extreme low summer flow of the rivers in some recent years has permitted seasonal intrusion of salt water from San Francisco Bay into this area. The productivity of another 400,000

U. S. Bureau of Reclamation Photo

PLACING CONCRETE LINING IN THE CONTRA COSTA CANAL

acres of rich lands is menaced here by saline encroachment which could be prevented by regulation of the river flow so as to provide constant fresh-water flushing of the delta waterways. This is another objective of the Central Valley Project.

The Sacramento and San Joaquin rivers are indicated in Fig. 1 by light lines, and the principal hyhraulic features of the Central Valley Project by heavy lines. The major structure is to be Shasta Dam, above Redding, which will create a storage reservoir with a gross capacity of 4,500,000 acre-ft, to be operated to stabilize the flow of the Sacramento River, and thereby diminish floods in the spring, eliminate extreme low flow in the fall, permit restoration of year-round navigation far upstream, provide constant flushing of the sometimes-salty waterways in the Sacramento–San Joaquin delta, and afford a surplus of water for export to the Contra Costa area and the San Joaquin Valley. Hydroelectric power generated at Shasta Dam will be carried down the Sacramento Valley over a 200-mile transmission line to the San Francisco Bay region.

FIG. 1. PRINCIPAL FEATURES OF THE CENTRAL VALLEY PROJECT

The Delta Cross Channel below Sacramento will facilitate the delivery of surplus Sacramento River water through the delta to the points of diversion of the Contra Costa Canal and the San Joaquin pumping system, while the 46-mile Contra Costa Canal, taking fresh water out of the delta west of Stockton, will serve an industrial and agricultural area in the upper Bay region. The 100-mile San Joaquin pumping system, taking

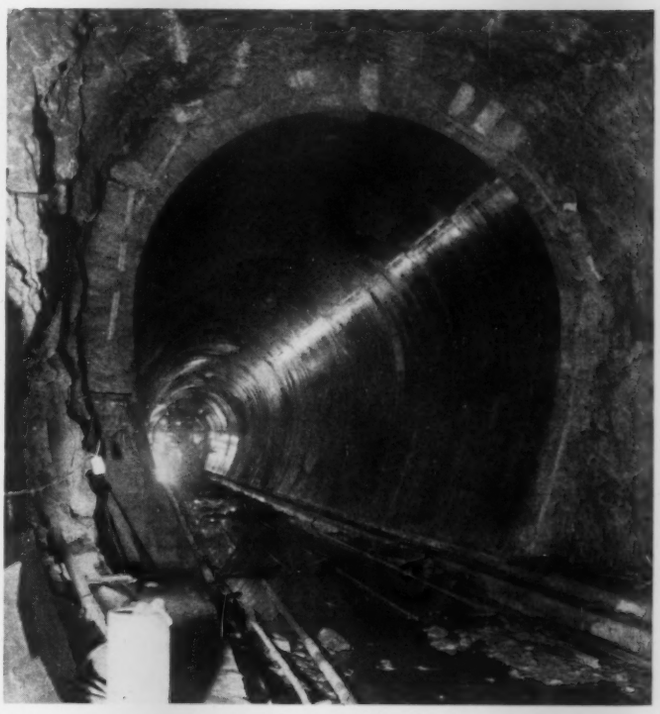
water out of the delta near Stockton, will carry it up the northern San Joaquin Valley as far as the bend in the river west of Fresno, to serve crop lands which now are using San Joaquin River water that is to be conserved and diverted at Friant Dam, near Fresno. From Friant Reservoir, which will have a gross capacity of 520,000 acre-ft, irrigation water will be diverted northerly 40 miles, via the Madera Canal, to thirsty lands of Madera County, and southerly 160 miles via the large Friant-Kern Canal to areas of critical water deficiency between Fresno and Bakersfield.

GENERAL DESCRIPTION OF SHASTA DAM

As designed by the Bureau of Reclamation, Shasta Dam is to be a slightly curved gravity-section concrete dam, about 560 ft high from the lowest foundation to the top, and 3,500 ft long on the crest, and about 580 ft thick at the base. A 375-ft overflow spillway section in the center is to be laid out on a tangent with flanking abutment sections curved on a 2,500-ft axial radius. Rolled earth and rock embankments with central concrete core walls are to be used instead of concrete gravity construction for short sections at either end, where the depth to sound rock is comparatively great. The completed dam and power house will contain more than 5,600,000 cu yd of concrete, and the dam will also contain 1,600,000 cu yd of earth and rock fill.

Thirteen steel-lined outlet conduits 102 in. in inside diameter will be installed in the spillway section to regulate outflow during periods of normal operation. Their combined capacity will vary from 11,000 cu ft per sec at maximum drawdown to 63,000 cu ft per sec with a full reservoir. Flood waters in excess of this quantity will pass over the central spillway (capacity 187,000 cu ft per sec), controlled by three 110 by 28-ft drum gates.

The power plant will be housed in a seven-story building adjoining the spillway on the right bank. Initial installation is to consist of four 75,000-kva generators, direct-connected to 103,000-hp turbines operating under heads of 238 to 475 ft. Contracts have been awarded for the manufacture of this equipment, and provision is being made for the installation of one similar additional unit in the future.



U. S. Bureau of Reclamation Photo

THIS BY-PASS TUNNEL AT SHASTA DAM IS NOW COMPLETED AND CARRYING MAIN-LINE RAILROAD TRAFFIC

Later On, the Construction Program Calls for Its Use to Divert the River Flow During Concreting Operations

A contract for the construction of Shasta Dam and power plant was awarded in July 1938 to Pacific Constructors, Inc.—a syndicate of 12 large contracting firms representing virtually every section of the United States—and work was started in September 1938. A general view of operations in the late spring of 1939 is shown in an accompanying photograph, taken from the top of the east abutment and looking toward the west abutment excavation. The white water tank in the left background marks the approximate future top of the dam. Most of the excavated material has been placed in terraced stock piles, seen at the right, for future use in embankment construction.

At the bottom of the picture, on the west bank of the river, may be seen the main line of the Southern Pacific railroad crossing the dam site. Late in June, traffic on this line was diverted through an 1,820-ft by-pass tunnel under the west abutment, and the old track was torn up to permit excavation for the dam to proceed on the lower west side of the canyon. When the permanent railroad relocation is completed and opened to traffic, this by-pass tunnel will be available for river diversion while concrete is being raised in the dam.

Railroad and highway relocation work in the vicinity of Shasta Dam is worthy of more than passing mention. The existing railroad out of Redding winds up the Sacramento River Canyon past Coram, the dam site, Kennett, and Pollock. The relocated 30-mile high-line now being constructed crosses

U. S. Bureau of Reclamation Photo

EXCAVATION IN PROGRESS ON THE WEST ABUTMENT OF SHASTA DAM

U. S. Bureau of Reclamation Photo

FIRST UNIT OF A 30-MILE RAILROAD RELOCATION AROUND SHASTA RESERVOIR
This Structure, Four-Fifths of a Mile Long, Is One of Eight Major Bridges Included in the Project

the Sacramento River just out of Redding and heads toward the Pit River east of the present route. The existing highway (U. S. 99) will be unchanged between Redding and the Pit River. The new railroad and highway both will cross the Pit River arm of the reservoir on a high double-deck bridge, and from there on will follow the ridge back to the Sacramento River canyon to rejoin the present routes near the town of Delta. The new railroad line includes 12 tunnels and 8 major bridges.

Of special interest is the huge Pit River Bridge, which will carry both the relocated railroad and the highway over part of Shasta Reservoir. It is believed that this structure will be the world's highest double-deck bridge, having a highway deck about 500 ft above the present water level of the river. It will be 3,590 ft long—including a highway approach of 210 ft at the southerly end; a main double-deck structure 2,754 ft long over the canyon; and another highway approach of 624 ft at the northerly end. The main structure will consist of two simple truss spans at the southerly end, three simple truss spans at the northerly end, and a three-span cantiliver structure in the center. The upper deck, accommodating four lines of traffic, will have a 44-ft highway flanked by $2^1/_2$-ft walkways on either side, and the lower deck will be occupied by two railroad tracks. The substructure will include two piers 350 ft high. This bridge, soon to be advertised for construction, will require 90,000 cu yd of concrete, 3,600 tons of reinforcing steel, and 16,400 tons of structural steel.

FRIANT DAM SOON TO BE STARTED

More than 300 miles from Shasta is the Friant Dam site on the upper San Joaquin River 20 miles out of Fresno. Bids for the construction of Friant Dam are to be opened by the Bureau of Reclamation in Sacramento on September 7. This structure will be about 300 ft high from the lowest foundation to the top, and 3,430 ft long on the crest. It will create a reservoir with a gross storage capacity of 520,000 acre-ft, of which 70,000 acre-ft is to be reserved for flood control and 316,000 acre-ft will be active irrigation storage. The dam will serve to divert water into the Friant-Kern and Madera canals which will carry supplemental irrigation water to parts of five counties.

Similar to Shasta in design, but with a straight axis, Friant will be a gravity-section concrete dam with an overflow spillway in the center. For normal operation, six river outlets through the dam will provide for a maximum discharge of 15,000 cu ft per sec. In addition, there will be installed four 96-in. needle-valve outlets to feed the Friant-Kern Canal (initial capacity, 3,500 cu ft per sec), and two 78-in. needle-valve outlets to feed the Madera Canal (capacity 1,000 cu ft per sec).

The Friant Dam spillway, which will be controlled by three 100 by 18-ft drum gates, is designed to pass a flood of 90,000 cu ft per sec with 18 ft of water going over the crest. In the spillway section the dam will be 260 ft thick at the base. Friant Dam will require 1,850,000 cu yd of concrete, about a third as much as Shasta.

Contracts awarded to date on the three principal features now under construction—Shasta Dam and power plant, the 30-mile Shasta railroad relocation, and the Contra Costa Canal—total about $60,000,000. The program soon will be extended to include Friant Dam and the San Joaquin Valley canals. One-third of the states of the United States are represented in construction work contracted so far, and three-fourths of the states have participated in the furnishing of materials and supplies. Approximately 3,000 persons are engaged on the project by the Bureau of Reclamation and its contractors. Federal funds made available to date total $44,600,000. Actual expenditures have reached $20,000,000 and are running almost $2,000,000 a month.

U. S. Bureau of Reclamation Photo

THE FRIANT DAM SITE, WITH BUREAU OF RECLAMATION CONSTRUCTION CAMP AT LOWER RIGHT

Protecting Chicago's Water Supply

Historical Sketch of Engineering Work, Legislation, and Litigation, from 1833 to 1938

By Samuel N. Karrick

Member American Society of Civil Engineers

Major, Corps of Engineers, U.S.A.; District Engineer, U. S. Engineer Office, Chicago, Ill.

THERE came a time around the middle of the last century when it appeared that Chicago was nearing the limit of its growth—choked by its own filth. Death lurked in the water, and wave after wave of cholera and typhoid swept over the town. How these conditions were conquered is a fascinating story of engineering achievement. And it is one in which chapters are still being written, for the ever-expanding city makes continual new demands on its engineers to keep pace. Major Karrick's article—a concise review of the protective measures of the past hundred years—is an abridgment of his paper before the Waterways Division at the 1939 Annual Convention.

LAKE Michigan, from which Chicago derives its water supply, is second in size of the Great Lakes of North America. Its water surface has an area of 22,400 sq miles, and its watershed, including the lake itself, is 69,000 sq miles. Over this basin the average annual rainfall is 32 in.

Separating the city of Chicago into three parts is the Chicago River, with its two branches—a comparatively small, sluggish stream. A ridge of limestone about 12 ft above lake level, running nearly parallel with, and 10 to 20 miles distant from, the shore of Lake Michigan, forms the divide which separates the watershed of the Great Lakes from that of the Mississippi River.

The first sanitary regulation for the river, the original source of water supply, was adopted by the village of Chicago on November 7, 1833, when the village trustees declared it unlawful to "throw or put into the Chicago River, within the limits of the town, any carcass of any dead animal or animals, under a penalty of three dollars for each offense." With growth of the settlement into a village, the river became increasingly polluted, and the inhabitants resorted to the lake and to shallow wells in the sand.

In 1836 the State Legislature granted a charter to the Chicago Hydraulic Company, a private enterprise, to supply the village with water. Its works, completed in 1842, consisted of an intake pipe enclosed by a pier extending about 150 ft into the lake at the foot of Lake Street, from which water was pumped into a wooden tank and discharged by gravity through primitive wooden mains of 2 to 6-in. diameter. At that time the city, which had been incorporated in 1837, had a population of 4,500.

Youthful Chicago did the obvious thing—discharged its sewers and ditches into the Chicago River. A small amount of sewage produced no appreciable effect upon the great volume of fresh water in Lake Michigan. An important aid in keeping the river in a sanitary condition was the Illinois and Michigan Canal, opened in 1848, extending from Chicago to La Salle, 96 miles distant on the Illinois River. In order to supply water to the summit level of this canal, the state constructed the "Calumet Feeder," from the Little Calumet River near Blue Island generally paralleling the present Sag Channel (Fig. 1), and erected a pumping plant at Ashland Avenue, where the Illinois and Michigan Canal left the South Branch of the Chicago River. The pumping plant, with a capacity of 400 cu ft per sec, was operated only eight hours a day during considerable periods, which proved insufficient to keep the river in a healthful condition. To remedy this situation, the City of Chicago obtained authority from the state to operate the pumps continuously and paid the extra cost out of its own funds. This arrangement prevented the polluted river water from discharging into the lake in dry periods, but during rainy periods the pumps were woefully inadequate for the purpose.

Pollution of the lake was rendered even more serious at times when the Des Plaines River overflowed its banks and discharged its flood waters through the west fork of the South Branch of the Chicago River, flushing the city's accumulated filth into the lake. Both cholera and typhoid fever were very prevalent in Chicago during 1849 and 1850.

In 1851 the Legislature granted the city a charter to build and operate its own water works, and in 1854 a new wooden intake, 3 ft by 4 ft, was extended into the lake about 600 ft at the foot of Chicago Avenue. The population of Chicago at that time was about 35,000.

At times this new water supply was not entirely satisfactory. The prevalence of typhoid and other waterborne diseases, resulting from the gross pollution of the lake by sewage, and by distillery, packing plant, and tannery waste, reached an alarming state. A Board of Public Works was created in 1861, charged with the responsibility of designing a new water supply, and two years later the city adopted the board's plan for a tunnel under the lake connecting with an intake two miles offshore at Chicago Avenue. The new system was placed in service in March 1867. A second tunnel was completed in 1874, and a third, connecting with the 4-mile crib off 14th Street, was placed in operation in 1892.

Subsequent to these installations additional intakes were constructed from time to time, the latest and largest being the recently completed 13-ft diameter tunnel extending from Chicago Avenue to a crib located immediately adjacent to the Carter Harrison Crib, 13,830 ft offshore (Fig. 1).

The water surface in the summit level of the Illinois and Michigan Canal, as originally constructed, was approximately 8 ft above mean lake level. In order that the volume of sewage carried away by the canal might be substantially increased, the city decided to lower this level at its own expense. This work was completed in 1871, at a cost of approximately $3,500,000. The resulting gravity flow in the canal varied between 600 and 700 cu ft per sec. This was quite satisfactory for a few years, but silting occurred more rapidly than expected, and during periods of low lake level there was little or no flow down the canal. Observations in 1879 showed that for 30 days the direction of flow was lakeward, followed by 10 days with no perceptible flow in either direction. The typhoid rate was again high.

It was therefore decided to build new pumps of a larger size at the entrance to the canal and resume the

use of the lock for navigation at that point. The new pumps, completed in 1881, had a capacity of 1,000 cu ft per sec and wrought considerable improvement. However, in 1885 a great flood occurred which reversed

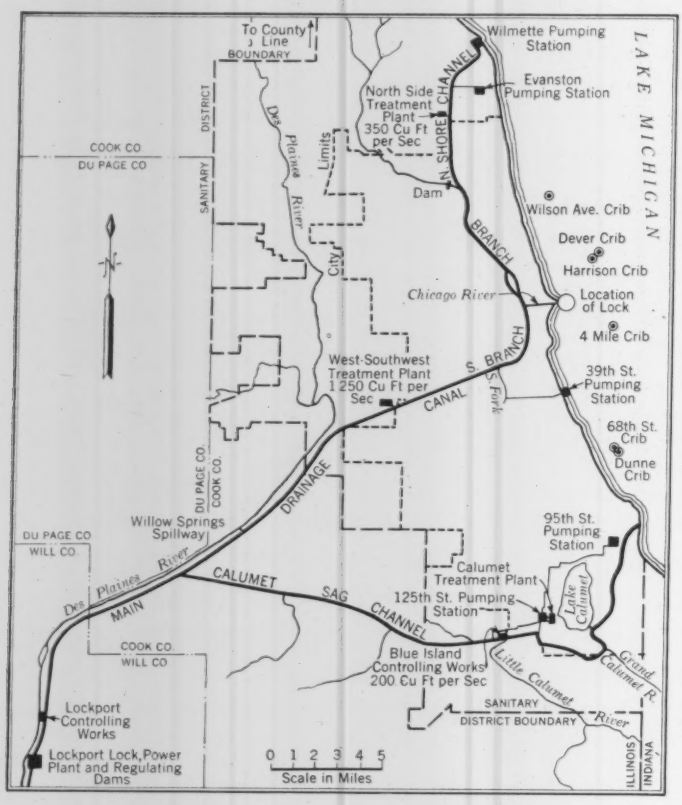


FIG. 1. INTAKE CRIBS AND CHANNELS, SANITARY DISTRICT OF CHICAGO

the flow in the Chicago River and carried great quantities of sewage and accumulated sludge into Lake Michigan, even as far as the Chicago Avenue intake crib. In the same year the city created a Drainage and Water Supply Commission to make a thorough, complete, and scientific investigation of the entire subject of water supply, sewerage, and drainage.

SANITARY DISTRICT ORGANIZED; CANALS BUILT

In 1887 this commission made a report in which it considered every known method of sewage disposal and also the growing demand for a deep waterway from Lake Michigan to the Mississippi River. It recommended as a complete solution of Chicago's difficulties the following:

1. That a sanitary and ship canal be constructed, connecting Lake Michigan at Chicago with the Des Plaines River at Lockport, of sufficient size and capacity to permit a flow of 24,000 cu ft per minute of water for each 100,000 of population, based on a total of $2^1/_2$ million people; and
2. That this canal be constructed to such depth and so connected with the Chicago River as to permanently reverse it and send its flow through the Des Plaines and Illinois into the Mississippi River.

The foregoing study led to the passage by the legislature of the Sanitary District Act of 1889, and the organization of the Sanitary District of Chicago. Construction of the main canal was started in 1892, and completed in 1899, at a total cost of approximately $31,163,000, exclusive of interest on bonds, and administrative, legal, and clerical expenses.

The main drainage canal extended 28 miles from the Chicago River at Roby Street to the old controlling works, consisting of a 160-ft bear-trap dam, above Lockport, Ill. The canal had a depth of 24 ft and a minimum width prevailing along the lower 15 miles, where the excavation was entirely in limestone rock, of 160 ft.

The main channel had been designed for a discharge capacity of approximately 10,000 cu ft per sec at mean lake level, which was considered sufficient to carry away the runoff from the heaviest rain that could be expected in the Chicago River watershed. It was estimated that during normal periods this quantity of water would properly dilute the sewage of three million people, the population which it was estimated would be reached by 1920.

Soon after the opening of the main canal, in order to secure the greatest head possible for the development of water power, the heavy embankments enclosing the canal were extended two miles farther downstream, where a new power house was constructed. The head available varies from 34 to 41 ft. Later, abreast of the power house, the State of Illinois constructed the first lock of the Illinois Waterway.

The Calumet River, which drains the highly industrial "Calumet area" in Illinois and Indiana south of Chicago proper, discharges into Lake Michigan about 12 miles south of the mouth of the Chicago River. In order to prevent contamination from this source it became necessary to reverse the direction of flow in this stream also. This was accomplished by the construction of the Calumet-Sag Channel (Fig. 1) which was begun in 1911 and completed in 1922, at a cost of approximately $14,175,000, exclusive of interest on bonds, and administrative, legal, and clerical expenses. The completed channel was 16.2 miles long, 20 ft deep, and 60 ft in minimum width. The flow is regulated by sector gates in the Blue Island Lock.

Other work carried out by the Sanitary District of Chicago has included the extension of the main channel through Joliet, the widening and deepening of the South Branch of the Chicago River, the construction of the North Shore channel, the construction of intercepting sewers, sewage pumping stations, and so forth. The total construction cost of the dilution system, not including the power house and electrical distribution features, was approximately $81,000,000.

RESTRICTIONS AND LITIGATION, 1899–1930

When the main channel was nearing completion, the Secretary of War, on May 8, 1899, issued an operating permit to the Sanitary District, which included the following condition:

"That if, at any time, it become apparent that the current created by such drainage works in the South and main branches of Chicago River be unreasonably obstructive to navigation or injurious to property, the Secretary of War reserves the right to close said discharge through said channel or to modify it to such extent as may be demanded by navigation and property interests along said Chicago River and its South Branch."

Subsequently, upon complaint of navigation interests, the Secretary of War restricted the discharge, and on December 5, 1901, issued a permit limiting the flow to 4,167 cu ft per sec throughout the 24 hours. This permit was in force until March 3, 1925, and was the one involved in the litigation between the United States and the Sanitary District of Chicago.

This litigation involved the question of the authority of the Secretary of War. In attempting to comply with the state law, the Sanitary District had exceeded the

limit of 4,167 cu ft per sec, and the Government filed suit in 1913 to enjoin the taking of more water than that authorized by the Secretary of War. The Supreme Court of the United States, in its decision of January 25, 1925, upheld the authority of the Secretary of War.

The Sanitary District thereupon filed application for a permit to divert 10,000 cu ft per sec from Lake Michigan, and on March 3, 1925, the Secretary of War issued a permit which authorized the diversion of 8,500 cu ft per sec in addition to the domestic pumpage, which was approximately 1,500 cu ft per sec. By 1925 the Sanitary District included all of Chicago and 49 other cities and villages, covering a total area of 437.4 sq miles.

In the litigation between the lake states and the Sanitary District, the former sought to restrain the Sanitary District from diverting any water whatsoever from the Lake Michigan drainage basin into the Mississippi watershed. The special master who heard the testimony recommended that the bill for injunction be dismissed, but the Supreme Court in an opinion rendered January 14, 1929, held that in the absence of affirmative action by Congress, the diversion, since it was for purposes of sanitation, had no legal basis and that the complainant lake states were entitled to a decree.

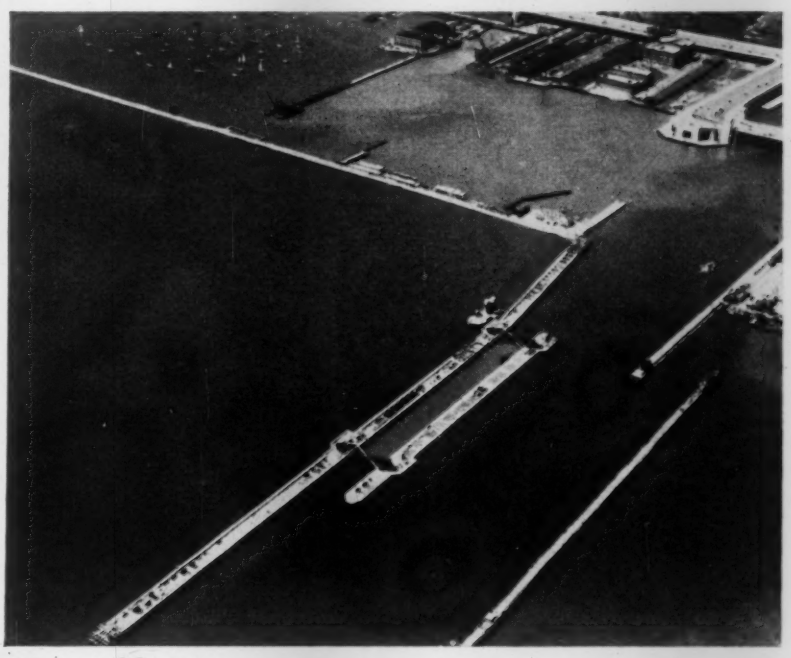
The case was then re-referred to the special master to determine what measures were necessary for the disposal of the sewage of the Sanitary District through means other than the diversion of water from Lake Michigan; to determine the time required to build the necessary works; to decide what reductions in diversion would be practicable from time to time, pending the completion of such works; and to determine what diversion would be necessary for maintaining navigation in the Chicago River under healthful conditions. The question of what quantity of water might be needed for the Illinois Waterway was excluded from consideration because the Illinois Waterway, from Chicago to Starved Rock, was not at that time a federal project.

The Supreme Court in its decree of April 21, 1930, enjoined the Sanitary District from diverting any of the waters of the Great Lakes watershed through the Sanitary Canal in excess of an annual average of 6,500 cu ft per sec, in addition to domestic pumpage; and specified that the figure of 6,500 was to be reduced to 5,000 at the end of 1935 and to 1,500 at the end of 1938.

The River and Harbor Act approved July 3, 1930, after authorizing the expenditure of $7,500,000 for completion of the Illinois Waterway, provided further that as soon as practicable after the waterway was completed, the Secretary of War should report to Congress his recommendations as to the minimum amount of flow that would be required to meet the needs of the waterway and that would not substantially injure the existing navigation on the Great Lakes. This report was prepared by the District Engineer in 1933 and published as H.D. 184, 73d Congress, 2d session. Two of the conclusions of the District Engineer read as follows:

"1. The diversion from Lake Michigan prescribed by the Supreme Court, plus the pumpage, is sufficient for the operation of the locks and dams now in use between Lockport and Starved Rock, as well as for those proposed in this report.

"2. The program of construction of plants for complete sewage treatment should be carried out, diversion should be reduced as required by the Supreme Court decree, and navigation conditions along the waterway and river should be observed for a short period subsequent to January 1, 1939. Then, and only then, can it be determined with reasonable certainty whether any additional diversion is necessary in order to provide decent and healthful living conditions for boat crews and river terminal operators. The improvements for navigation herein proposed must be completed before January 1, 1939."



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CONTROLLING WORKS DURING CONSTRUCTION—COMPLETE EXCEPT FOR THE WALL JOINING THE FAR END OF THE LOCK WITH THE PIER TO THE RIGHT

After the opening of the main canal, reversals of flow in the Chicago River became so infrequent and of such short duration that an unjustified sense of security had been created. A rude awakening occurred on August 11, 1923. During the night a very heavy rain fell throughout the city. The canal was being operated in the customary manner, with approximately 11,500 cu ft per sec passing through the turbines, and as daylight dawned, part of the turbines were closed down. Soon after daybreak the bridge-tenders along the Chicago River discovered that the river was running toward the lake, carrying with it into the city's drinking water an enormous amount of indescribable filth which had been flushed out of the North Branch. Nothing was done to stop the lakeward flow of filth until nine o'clock when the Sanitary District officials, learning of the situation, telephoned to Lockport to operate all of the generators and open the bear-trap gates; however, the flow toward the lake was not checked until noon. Fortunately, a strong wind from the south drove most of the surface material to the north, where it was trapped inside of the outer breakwater. Only a small amount of pollution passed through the main channel opening in the outer breakwater.

Had there been proper coordination between observers in Chicago and operation at the Lockport power house, it would have been possible to greatly reduce the period of reversal, by temporarily increasing, rather than decreasing, the flow at the power house during the early morning hours. Immediate steps were taken to remedy this condition, and the few reversals that have occurred

since have not been of long duration. However, the conclusion was reached by the U. S. Engineer Office and concurred in by the Sanitary District engineers that to prevent such reversals under all conditions, a flood gate or

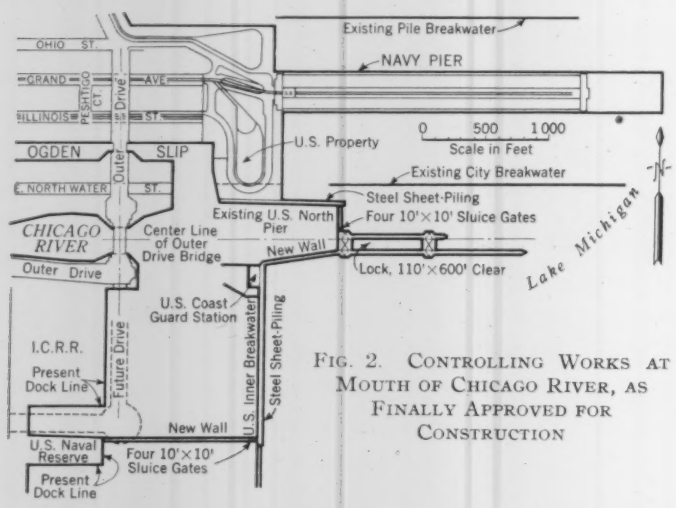

FIG. 2. CONTROLLING WORKS AT MOUTH OF CHICAGO RIVER, AS FINALLY APPROVED FOR CONSTRUCTION

controlling works should be built in the Chicago River, near the mouth. In the War Department permit of March 3, 1925, one of the conditions accordingly provided that the Sanitary District should construct such a gate to be ready for operation by July 1, 1929.

Following the issuance of the permit, the Sanitary District and the U. S. Engineer Office cooperated in the preparation of various plans for such works. The plan that originally met with the greatest favor was a steel pontoon gate hinged at one corner to one of the abutments of the Outer Drive Bridge. Normally the pontoon would have occupied a recess in the north dock wall, leaving an unobstructed navigation channel 220 ft wide between the bridge abutments. Whenever there was danger of a reversal, the pontoon would have been swung across the channel and lowered to a substantially watertight connection across the bottom of the river.

These plans encountered Department objections on the grounds that such a flood gate when closed would completely bar navigation between the lake and the river. The District Engineer was accordingly requested to give consideration to plans that would assure uninterrupted navigation, and in 1928 he submitted general plans for a standard concrete lock to be located at the mouth of the Chicago River, and also a plan which utilized the existing dock walls for lock walls, using the full width of the river as a lock chamber. Following this, on request of the Chief of Engineers, a comprehensive report was prepared including plans and estimates for all feasible methods of preventing reversals of flow.

This report was submitted by the District Engineer on March 11, 1929. Plans were developed for utilization of a lock at any one of three different locations. Another scheme contemplated construction of a lock, with Tainter gates, in the Sanitary Canal, about $1\frac{1}{2}$ miles below its head; this would leave the Chicago River free of all hindrances to navigation, but had the disadvantage that some polluted water would escape into the lake before reversal could be accomplished by opening the gates at Lockport. Many other plans were also considered. However, objections were presented to all these proposals—some by the Sanitary District, some by other local interests, some by the Engineer Department, and some by navigation interests. As a result, the Chief of Engineers withheld his approval of any of the plans advanced.

This was the status on December 19, 1932, when upon complaint of the other lake states, the Supreme Court appointed a special master to determine the causes of the delay. The hearing was commenced the following month, but for some time following it nothing was done. The Sanitary District realized that it would be necessary to build a large lock at some location, but was without the necessary funds.

Early in May 1935, engineers of the Sanitary District of Chicago called on the District Engineer to discuss the matter of controlling works. It was believed by all that a lock at the mouth of the river was the most desirable solution, particularly at this would make it possible to keep the water in the Chicago River at or near −2.0, Chicago City datum, thus securing a maximum vertical clearance under the Chicago River bridges, as well as those along the Sag Channel and the main canal of the Sanitary District, and would be the least obstructive to navigation. Thereafter, on September 21, 1935, the Sanitary District secured from the Public Works Administration an allotment of $3,000,000 for the construction of such works.

The plans recommended by the Sanitary District called for two concrete walls across the river, a short distance east of the Michigan Avenue Bridge and 800 ft apart. Each wall was to be provided with a pair of mitering lock gates giving a clear opening of 110 ft. As the head would never be very great, the existing bulkheads along the banks of the river were to be used for lock walls. It was proposed to use this lock only when there was a tendency for the river to flow into the lake.

Adjacent property owners registered strong protests against this plan and gave notice that they would demand very large damages; other interests also objected. Two subsequent proposals met with similar disfavor, and finally the Chief of Engineers authorized the District Engineer to advise the Sanitary District that favorable consideration would be given to the substitution of a complete lock 80 by 600 ft in the clear, located about 1,800 ft east of the Outer Drive Bridge, the lock walls to be parallel with the Navy Pier, and connected with existing structures.

LOCK PLANS FINALLY APPROVED; WORK COMPLETED

On February 8, 1936, the Sanitary District submitted plans in accordance with these suggestions (Fig. 2). These plans provided for steel sheet piling along the east half of the U. S. North Pier and along the north 1,600 ft of the U. S. Inner Breakwater, to secure watertightness and prevent polluted river water from entering the harbor. As all the features previously objected to had been eliminated, another public hearing was unnecessary. Such civic bodies as were interested in the matter considered the new plans excellent and suggested no change. A permit for construction was accordingly issued on March 12, 1936, subject to the provision that the works should be maintained and operated by, and at the expense of, the Sanitary District, under the general supervision of the District Engineer and subject to such regulations as might be prescribed by the Secretary of War.

On August 11, 1936, construction was started, and on September 7, 1938, the lock was officially opened to navigation. It has proved satisfactory to date, and all concerned believe it will fulfill the twofold purpose of sanitation and navigation for which it was designed.

Credit for compiling the data presented in this paper is due to John W. Woermann, M. Am. Soc. C.E., formerly Principal Engineer (Civil), now retired and employed by the U. S. Engineer Office, Chicago, Ill., in a consulting capacity.

Sheet-Pile Cofferdams and Test Cell for TVA Projects

Seventh of a Series on Modern Construction Tools and Practice

By R. T. COLBURN, W. AM. SOC. C.E.

CONSTRUCTION PLANT ENGINEER, TENNESSEE VALLEY AUTHORITY, KNOXVILLE, TENN.

EXPERIENCES of the Tennessee Valley Authority with circular-cell sheet-pile cofferdams constitute Mr. Colburn's second contribution to this series of articles. By text and pictures he describes the construction and demolition of such structures at three of the TVA's dams, and the tentative plans for similar work at a fourth location. The use of freezing to aid in the excavation of a deep, sheet-pile-lined test cell is also discussed. This article is part of a paper on the program of the Construction Division at the Society's 1939 Spring Meeting.

ONE of the major construction operations on the lower Tennessee River projects was the building of cofferdams. The Pickwick Landing, Guntersville, and Chickamauga projects have all used the circular-cell sheet-pile type of design. Figure 1 is a view of the lock cofferdam at Pickwick Landing Dam. This cofferdam had a head of 55 ft, measured from the top of the cofferdam to the base of the sheet piling, which rested directly on the solid rock. The cofferdam area was excavated completely down to rock, no inside berm being provided. It is believed that this was the highest cofferdam of the circular sheet-pile cell type ever designed without a berm or some other from of support on the inside. It was fully tested by a flood which came within a few inches of the top, and no distress was evident. Weep holes cut in the inside cell walls were intended to reduce the internal pressure resulting from saturation of the cell.

A test was made to provide a check on the saturation line by means of a row of pipes driven vertically into a cell across a section of the cofferdam. The results of the tests indicate that although provision was made for draining the cell, the saturation line was nearly horizontal over about three-quarters of the cell and only a few feet below the level of the river, the exact amount at any particular time depending on the rate of rise or fall of the stream. When the water rose to the top of the cofferdam for several days the entire cell became saturated. The weep holes probably resulted in partially reducing the internal pressure even under this condition, but did not prevent saturation.

The cofferdams were built with standard 15-in. piling with $^3/_8$-in. web, and were filled with sand and gravel. A theoretical interlock stress of 8,000 lb per lin in. of sheet pile was used as a working stress on the Pickwick Landing cofferdam.

The method of erection as well as the design was given careful study. The method used was developed by J. E. Walters, superintendent at Pickwick Landing Dam, and proved so successful that it was later used on both Guntersville and Chickamauga dams. Figure 2 shows the principal feature of this method—a circular templet equipped with spuds, which was handled and set in place by two skid derricks mounted on barges. This templet was built of pipe, welded together and suitably braced, providing a strong structure light enough to be handled by the floating derricks. The operation consisted of setting the templet in place and anchoring it by means of the spuds, setting the piling around the templet with the derricks, and then driving the piling evenly through the overburden to rock. With completion of the cell the templet was lifted out and placed in position for the next cell. The completed cell was filled with gravel and sand by a hydraulic dredge.

Among the reasons for using circular cells instead of the arc diaphragm type were the following: There was less construction hazard due to the fact that they could immediately be filled; the circular templet was easier to handle than that required for the arc diaphragm type; and for this height of cofferdam, there was no saving in the amount of sheet piling by the use of the arc diaphragm type.

The same sheet piling was re-used on each of these projects for each of the three successive cofferdam stages. Pile pulling was done in some cases from the floating rigs and in some cases from a crane mounted on top of the cofferdam. In most cases, a pile extractor of the standard type was all that was required. However, at Pickwick Landing Dam, where it was necessary to provide a greater pull, the arrangement shown in Fig. 3 was used. With this arrangement of sheaves and extractor attached to two cranes, a total pull of 175 tons was possible. The cell was partly excavated before pulling operations began.

Driving performance on lock cofferdams for the three projects has been as shown in Table I. The average costs for driving and pulling piling at Pickwick Landing Dam for all three cofferdam stages were as follows:

Handling and hauling	$1.405 per ton
Templets	2.541 per ton
Setting piling	5.602 per ton
Driving	3.639 per ton
Pile pulling	6.279 per ton

FIG. 1. LOCK COFFERDAM AT PICKWICK LANDING DAM

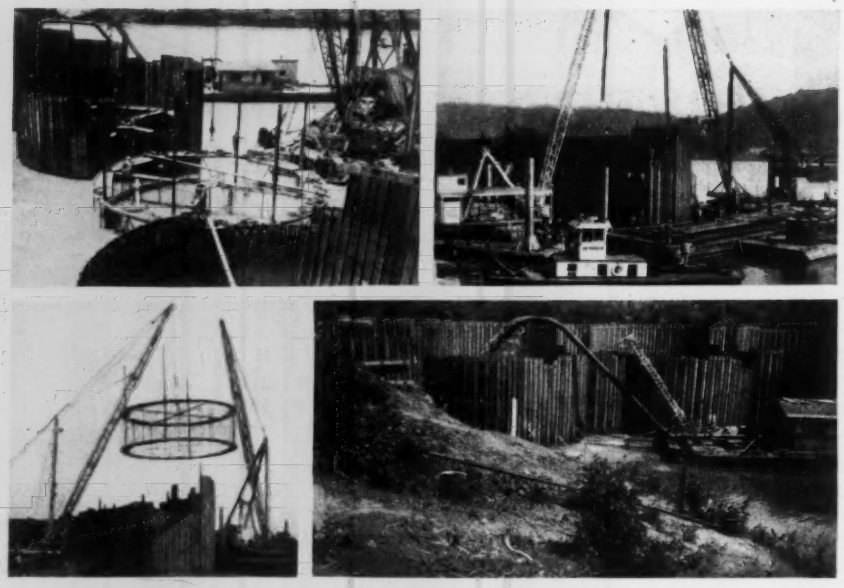

FIG. 2. COFFERDAM ERECTION METHODS
Upper Left, Templet in Place; Upper Right, Setting Piling for Cofferdam Cell Around Templet; Lower Left, Transferring Templet from Completed Cell to Position for Starting Next Cell; Lower Right, Filling Cells by Dredge

The total number of tons driven and pulled in three stages was approximately 13,800. The cost figures include labor, equipment operation and depreciation, and miscellaneous expense, but do not include cost of piling.

Discussion of the projected Gilbertsville cofferdam is somewhat preliminary at present as the final design is still in process of being worked out by the TVA central Construction Plant Division office. The tentative cross-section is shown in Fig. 4. It is interesting to note that this cofferdam will be from 90 to 100 ft high, and the cofferdam for each stage will be approximately 2,700 ft long, enclosing an area of approximately 22 acres. This is a pioneer design owing to its magnitude, the only known precedent for it being at Grand Coulee where a section of cofferdam was 90 ft high. There is at present a cofferdam of similar design 85 ft high on the Delaware River aqueduct project at the Kensico Reservoir in New York which is very nearly comparable.

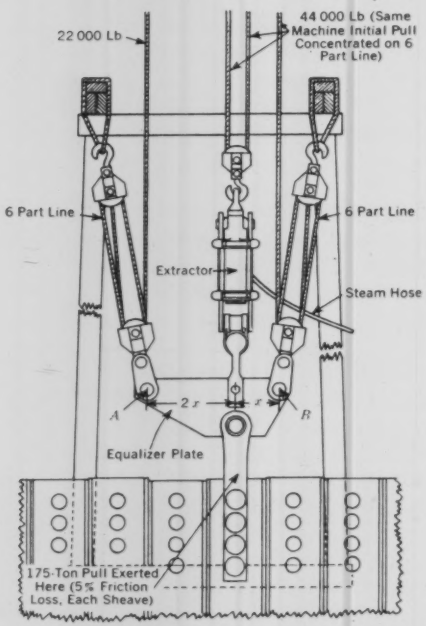

FIG. 3. ARRANGEMENT OF SHEAVES AND EXTRACTOR FOR PULLING PILES

Designs are being carefully studied and extensive tests of fill material and photoelastic model investigations have been made. It is planned to drive 70, 75, and 80-ft lengths of piling, which will constitute the lower three-quarters of the cells, and piece out the top with short lengths to the required height. This will bring the splice above normal water level so that the lower section of piling can be pulled. The fill material will be sand and gravel dredged from the river bottom. The long-length piling and the presence of over 40 ft of overburden in the bottom of the river involves some special construction methods which were not present in the earlier TVA cofferdams. The exact construction methods have not yet been completely worked out.

On all projects where sheet-pile cofferdams have been used, the river diversion scheme and construction stages have been first checked and tested in the TVA hydraulic laboratory at Norris. Figure 5 shows such a model test being conducted for the Pickwick Landing cofferdam. Velocities of the water in the restricted river channel in each stage were observed, and the effects of increased velocity on navigation and bank erosion, and in some cases bridge-pier erosion below the dam, were determined. The effect of rounding the corners of the cofferdam to produce an even stream flow was studied, and the cofferdam plan thus established. Tests of several possible river diversion schemes were made to confirm the theoretical analysis, and such studies were the basis for determining the river diversion scheme best suited to the construction program.

A unique construction operation was conducted at Gilbertsville Dam a year ago while sinking a 20-ft diameter test shaft 95 ft through the overburden to bedrock. This shaft was undertaken in order to study the effect of driving sheet piling 95 ft through overburden, to observe the condition of a fairly large section of bedrock,

TABLE 1. DRIVING PERFORMANCE ON LOCK COFFERDAMS

LOCATION OF LOCK	TOTAL SQUARE FEET DRIVEN	AVERAGE LENGTH OF PILING	AVERAGE DEPTH OF OVER-BURDEN	DIAM. OF MAIN CELLS	AVG. NO. HRS. PER CELL FOR CONSTRUCTION*
Pickwick	388,988	51′	14′	58.89′	21
Guntersville . . .	240,654	39.8′	6′	42.97′	19
Chickamauga . . .	288,880	43′	13′	47.75′	29.44†

* For setting templet, setting and driving piling, removing templet, moving equipment, and delays.
† At Chickamauga Dam the rock bottom was extremely uneven and in some cases piling was driven 30 ft below the normal rock bottom, which made it necessary to do a considerable amount of splicing. This difficult condition accounts for the longer time required to complete the cells on this project.

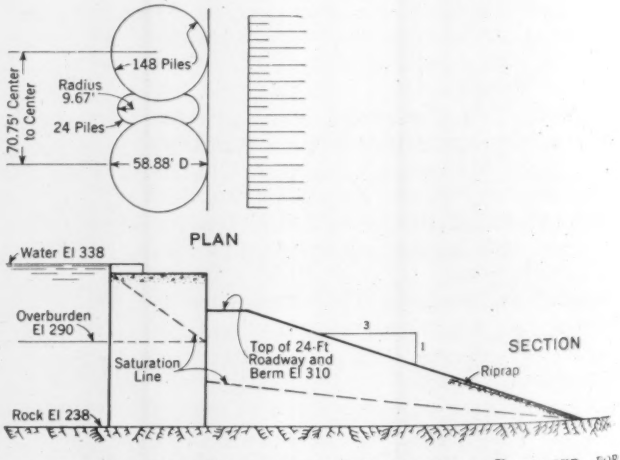

FIG. 4. TYPICAL SECTION OF COFFERDAM PROPOSED FOR GILBERTSVILLE DAM PROJECT (PRELIMINARY STUDY)

to provide an opportunity for diagonal drilling from the base of the shaft, to observe the effect of grouting, and to determine the type of overburden. Figure 6 is a photograph of the interior of this shaft when it was partly excavated.

Sheet piling was driven to rock, and as the excavation proceeded, bracing rings were added. With considerable difficulty the excavation was carried to within about 12 ft of the bedrock. At this depth the water pressure, aided by a number of leaks and some separations which occurred in the interlock of the piling caused a serious blow and prevented further progress by this method.

Grouting the bedrock with cement proved very satisfactory. Overburden grouting with cement and bentonite proved partially successful and the leakage was greatly reduced, although the percentage of effective grout to the total amount used was very small. After the blow it became evident that grouting the overburden could not be counted on to complete the shaft. The freezing of an earth embankment to prevent slides at Grand Coulee suggested the idea of freezing a ring around the perimeter of the shaft and sealing it into bedrock. Refrigeration units, including ammonia compressors and auxiliary equipment, were available for use from TVA-owned Muscle Shoals properties, and the cost involved was only that for installation and operation.

FIG. 5. HYDRAULIC MODELS PLAY A PART IN SHAPING CONSTRUCTION PLANS

Here the Surface Currents and Velocities at Pickwick Landing Are Being Investigated, with the "Second Stage" Cofferdam in Place

One freezing unit with a capacity of 23 tons of refrigeration, or the equivalent of 6,700,000 Btu in 24 hours, was placed in operation on February 9. This required about 50 hp to carry the load, but a gasoline engine of somewhat larger size was used because it was available. Figure 7 shows the top of the shaft with the arrangement of headers and freezing wells. Figure 8 shows a typical section of a freezing well. This well consisted of a 4-in. pipe casing driven to rock. Inside this casing a hole was drilled into the rock about 3 ft. A 2-in. pipe closed at the bottom and containing within it a 1-in. pipe open at the bottom were connected at the top to the brine exit header and the brine inlet header, respectively. The brine circulated down through the 1-in. pipe and back up the 2-in. pipe. These wells were spaced about 30 in. apart around the circumference of the shaft. The freezing process was delayed considerably by the rise and fall of the river, with the resulting inflow of warm water and outflow of cold water from the shaft. On March 28 it was decided to install an additional freezing unit, which was placed in operation on April 20, doubling the capacity of the plant.

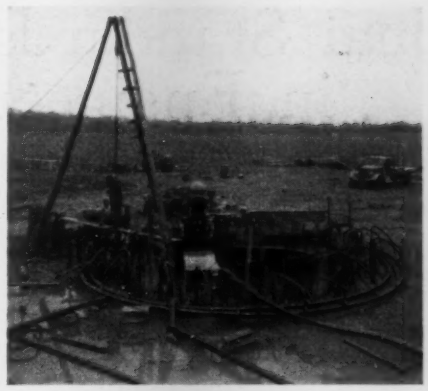

FIG. 7. TOP OF TEST SHAFT AT GILBERTSVILLE, WITH HEADERS AND FREEZING WELLS IN PLACE

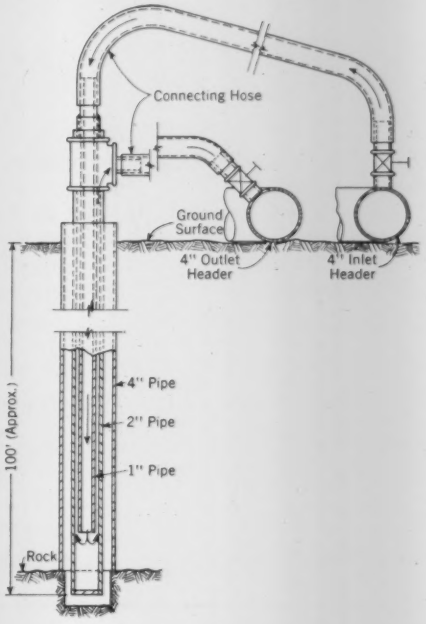

FIG. 8. TYPICAL SECTION OF FREEZING WELL

It appears from experience that the amount of refrigeration required should be considerably more than theoretically calculated, in order to provide against unforeseen difficulties such as the rise and fall of ground water. With the two units in operation, freezing progressed sufficiently to permit unwatering the cell on May 16. The excavation was then carried to bedrock, and observations and drilling were completed.

The coating of ice on the interior of the piling in the cell was about 18 in. thick, and it was observed that the thickness of the frozen overburden around the outside of the cell probably averaged 3 ft. The operation was carried on under direction of E. A. Prokop, construction engineer, and under supervision of B. W. Goodenough, senior engineer, and R. F. Rhoades, associate geologist.

Parenthetically, another application of refrigeration to construction purposes may be mentioned. At Hiwassee Dam, a freezing unit similar to those installed at Gilbertsville is being used to cool the mixing water for concrete. In warm weather this water is brought from between 80 and 85 F down to between 40 and 50 F. This reduces the internal heat in the concrete and is believed to reduce the number of shrinkage cracks in the completed structure.

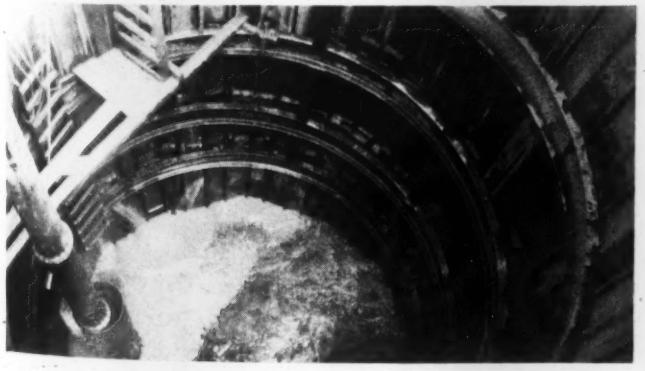

FIG. 6. LOOKING DOWN INTO TEST CELL, GILBERTSVILLE DAM

Professional Standards vs. Mass Production in Engineering Schools

By GEORGE C. ERNST, ASSOC. M. AM. SOC. C.E.

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THE professional status of engineering, and the educational policies, staffs, and graduates of the engineering schools are being subjected to a close scrutiny by the professional engineer with a view to correcting certain existing evils in the profession. Such activity merits more than a casual interest on the part of the instructional staffs.

Certainly the objectives of a professional school should be (1) the development of a true professional attitude in those (and only those) capable of maintaining the highest of professional standards, (2) professional growth of the teaching staff, and (3) thorough instruction in the technics of the profession, which should be distinguished from mere routine drill in the so-called fundamentals.

Mass production in education, accomplished by large classes, heavy teaching loads, or a combination of the two, will necessarily bring teaching to a lower level than that demanded in item (1) above. Professional growth of teaching staff (item 2) under such conditions will be of doubtful quality, while the technics of the profession (item 3) will rarely be touched upon in the routine of the classroom. Frankly, a profession should be little concerned with the quantity or quality of material temporarily absorbed by the students under such conditions. Even if complete attainment of instructional aims is assumed, it is inconceivable that a professional attitude would develop in every individual under the conditions of mass instruction. It is true that engineers of professional grade have been and can continue to be produced by methods of mass instruction, but the graduation of a uniformly high-grade product (as should be demanded by a profession) is certainly impossible.

Approximately a decade ago the S.P.E.E. (Society for the Promotion of Engineering Education) published the comprehensive report of "The Investigation of Engineering Education" (conducted from 1922 to 1933), containing a thorough analysis of personnel and curricula, with certain recommendations with regard to each which would allow for student instruction and staff development of a professional grade. Quoting from this report (Vol. 1, page 292) of ten years ago, "The time seems to be ripe for lessening the burden on the individual teacher of the formal undergraduate teaching and for an increase in the time devoted to more advanced instruction, to engineering practice, research, and to whatever tends to enrich and develop the teacher as an engineer and a man."

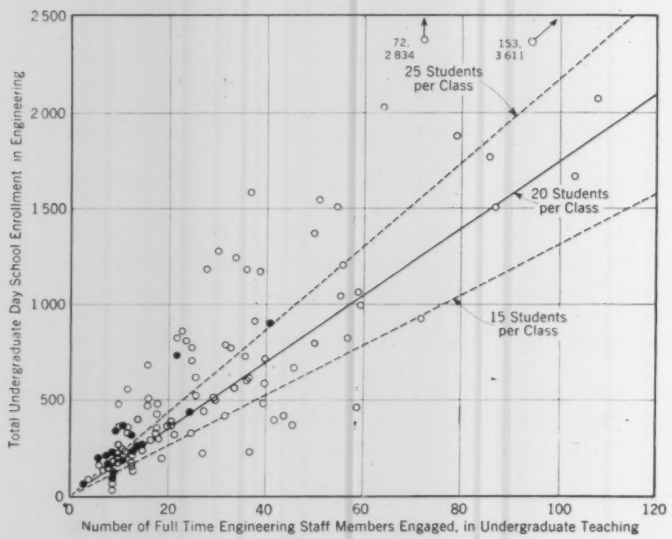

FIG. 1. ENROLMENT VS. STAFF FOR 110 ENGINEERING SCHOOLS
Open Circles Indicate E.C.P.D.-Accredited Institutions

Figures 1 and 2 merit attention in this respect. In Fig. 1, the teaching staffs of 110 institutions (as recorded in the latest catalogs), taken to include the grades (exclusive of those engaged solely in research) of professors, associate and assistant professors, instructors, associates, assistants, and 50 per cent of the teaching graduate assistants, have been plotted against total undergraduate day-school enrolments for 1938–1939 as recorded in the *Journal of Engineering Education* for January 1939. The S.P.E.E. report on engineering education recommended a teaching load of 15 classroom hours per week and gave data resulting in an average total semester hour credit for the students of 82.5 for engineering subjects. The average number of laboratory hours per staff member, as stated in the report, was equal to slightly over 10, which would produce an average credit hour load of 9 (based upon 3-hour laboratories) to correspond to the 15 classroom hours. No recommendations or existing averages for the exceedingly important factor of the number of students per classroom or credit hour is available in the report. Published information on this subject appears to be non-existent except for a thorough discussion several years ago by Professor L. O. Stewart, Assoc. M. Am. Soc. C.E., of Iowa State College (*Proceedings*, S.P.E.E., Vol. 42, 1934, page 225—Journal No. 3, 1933). Except for a very few institutions, the methods of handling class instruction do not differ materially, and the average number of students in each class for various schools should be approximately constant for equal teaching loads. Figures most frequently stated verbally are 12 to 16 for experimental or surveying laboratories, 15 to 20 for problems courses, 25 to 30 for purely lecture courses, and 20 to 25 for drafting periods. The larger figure in each case might be considered somewhat high for effective professional training. The solid straight line in Fig. 1 is plotted for 9 credit hours per full-time staff member, 82.5 semester-credit hours of engineering per student, and an average of 20 students per credit hour (or classroom meeting).

In Fig. 2, the 110 institutions have been ranked according to the number of student-credit hours per instructor per term calculated by using an average of the total engineering credits (from the catalog of the school) required in the curricula in chemical, civil, electrical, mechanical, and mining engineering. Although enrolments in the various curricula vary and additional curricula are offered in some schools, the diagram corrects for major differences in the quantity of engineering required by various institutions. It may be seen that the adjustment of such differences does not compensate for the scattering shown in Fig. 1. Limitation of staff-enrolment ratios would narrow this scattering markedly

It is evident that the majority of staff members must be carrying more than 15 classroom hours per week (or 9 credit hours) on the average, or the average number of students per classroom (or credit) hour is in excess of 20, or, as is generally the case, a combination of the two has been accepted as an unavoidable condition.

Grouping of the staff facilities is shown in Fig. 2 with 135 student-credit hours per instructor taken as the upper limit for the preferable group, 135 to 180 for the limits of good staff facilities, 180 to 225 for acceptable, and above 225 as questionable. For these limits, 32 per cent of the publicly supported schools fall in the upper two groups, while the percentages for the private and denominational are 60 per cent and 38 per cent, respectively. Accurate ranking, of course, should be based upon actual student classroom hours computed from existing teaching schedules sent in each term to an accrediting committee. This would be particularly important in institutions providing engineering instruction to certain non-engineering groups. A comparative study of the teaching loads carried in the professions of architecture, medicine, law, and engineering would be a worth-while investment if made to include differences in method of instruction. With continued inspection and accrediting by the E.C.P.D. committees, the general deviation from an appropriate instructional load, as well as the marked lack of uniformity among the institutions as a group, demands serious consideration.

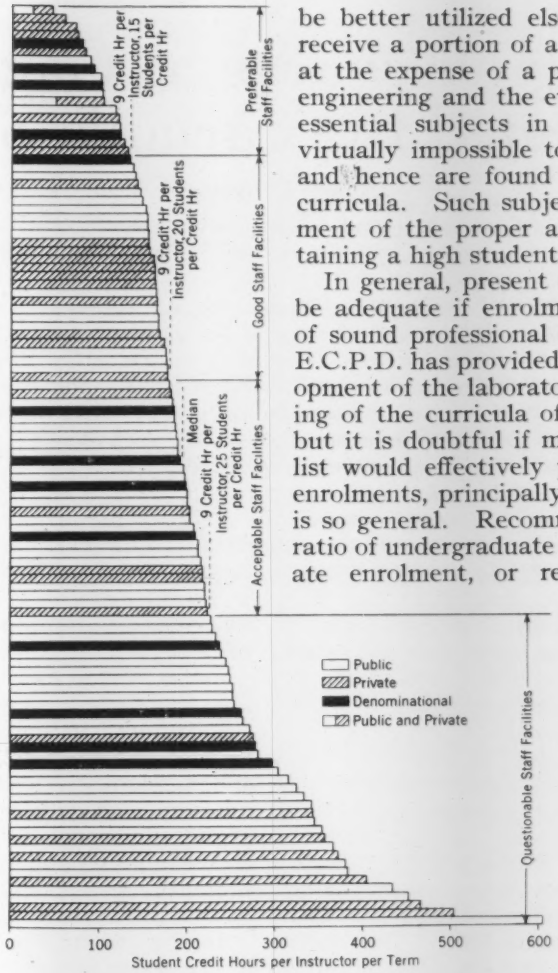

FIG. 2. RANKING OF 110 ENGINEERING SCHOOLS WITH RESPECT TO UNDERGRADUATE DAY-SCHOOL TEACHING LOAD

It should be apparent after ten years that to control the ratio of enrolment to size of staff within limits that will insure instruction of a professional grade is virtually impossible of accomplishment by an administrative head from within an institution supported by the public or operated for monetary return. Such control can come only from an outside accrediting source and, as such, would be evidence of professional solidarity. With only a few outstanding exceptions, the apparent academic policy has been (*a*) the general acceptance of practically all material offering itself for engineering education, and (*b*) the recommendation of engineering education as an "excellent training for any type of future work."

It is exceptionally disheartening to attempt the instruction of students who admittedly are "taking engineering because it is a good 'general' training." Such students see no point in the development of ideals commensurate with those required by a profession since they have no intention of becoming a member of one. The majority of the graduates, however, seem to expect work of an engineering nature after graduation. With regard to those who do not graduate (approximately 60 per cent of those entering), it would appear that much time has been spent by staff and students which would be better utilized elsewhere. Such students certainly receive a portion of a "liberal" or "general" education at the expense of a professional staff. The history of engineering and the ethics of professional practice (two essential subjects in any professional education) are virtually impossible to "get across" in mass education and hence are found in a relatively small number of curricula. Such subjects, however, aid in the development of the proper attitude and would help in maintaining a high student morale.

In general, present engineering teaching staffs would be adequate if enrolments were restricted to students of sound professional quality. The accrediting by the E.C.P.D. has provided a powerful influence in the development of the laboratory facilities and in the strengthening of the curricula of the smaller engineering schools, but it is doubtful if mere omission from the accredited list would effectively produce a selective restriction of enrolments, principally because the need for restriction is so general. Recommendations as to the appropriate ratio of undergraduate teaching staff to total undergraduate enrolment, or recommended limits for average student classroom hours per staff member, however, would be of great aid, if not considered solely as a recommendation for enlargement of staff.

The symposium of the Committee on Professional Objectives of the American Society of Civil Engineers and the subsequent formal discussions (March and April 1939 issues of CIVIL ENGINEERING) suggest the desirability of a 5- or 6-year period of training. Such a training, if uniform throughout the institutions in this country, would effectively limit the enrolment as well as supply much needed additional time in the professional years. This objective is not likely to be accomplished by purely voluntary means. Those states with license laws could aid materially by legislation requiring a 2-year liberal arts training of specified quality and 3 or 4 years of engineering training before a license to practice is granted. In order to provide the necessary influence, such a movement would have to be sponsored by the technical societies acting as a unit. A combination of recommended limits for staff-enrolment ratios (or average student classroom hours per staff member) from an accrediting source, and the requirement of a 5-year course to obtain a license, would be a long step toward improvement of professional status.

SUMMARY

1. The objectives of a professional education, as stated in this paper, are not compatible with mass production in education.

2. In general, teaching loads are not within the limits that will allow professional development of staff and students.

3. Recommended limits for the number of students per classroom hour per instructor are seriously needed by the engineering colleges for control of the size of classes.

4. Limitation of staff-enrolment ratios or student-classroom hours per instructor, combined with a 5 or 6-year course prior to obtaining a license to practice, is needed if the status of the profession and the graduates and teaching staffs are to be improved.

Determining Discharge at Gaging Stations Affected by Variable Slope

By M. C. Boyer

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Associate Engineer, U. S. Geological Survey, Washington, D.C.

THE elements that ordinarily control the stage-discharge relation at a section on a natural stream may under some conditions of stage or discharge become inoperative. For example, the stage-discharge relations at gaging stations on tributaries near their confluence with other streams may be affected by high stages on those streams, or the relations at gaging stations above artificial structures, as dams, locks, or weirs, may be affected by their operation. These effects, which express themselves in variations in the water-surface slope, are known as backwater.

IDEALLY, stream gaging stations should be located at points where the stage-discharge relation is unaffected by changes in conditions downstream—that is, at points above the influence of backwater. As a practical matter, such locations are becoming increasingly difficult to find. Satisfactory results can still be obtained, however, by making use of simultaneous readings from two stations. In the accompanying paper, Mr. Boyer outlines the methods used by the Geological Survey in such cases. His contribution was on the Hydraulics Division program at the 1939 Spring Meeting.

In the earlier years of the Geological Survey, if the stage-discharge relation at a gaging station was affected by backwater it was often possible to relocate the station and to escape such effects. However, as the streams in the United States came to be developed for navigation, power, irrigation, and other purposes, many gaging stations that came within the influence of backwater could not be relocated above that influence. The determination of the discharge under such conditions required a study of variations in the slope of the water surface, and the development of relations between slope, stage, and discharge. It is the purpose of this paper to present an outline of the methods now used by the Geological Survey for that purpose.

Early students of the effect of variable slope on the stage-discharge relation attempted to adjust the discharge measurements for the effects of variable slope by use of the Chezy equation, $V = C\sqrt{RS}$. But since this equation was developed for steady flow in a uniform channel under conditions of complete turbulence, it was found to be not entirely suited to the problems of flow in natural channels. Thus it was necessary to turn to the equation for the slope of the energy gradient,

$$S_e = \frac{V^n}{KR^a} \quad \ldots\ldots [1]$$

in which V is the average velocity in a cross-section (determined by dividing the discharge by the cross-sectional area); K is an expression of the rugosities of the channel; R, a measure of the channel shape; n, a measure of the degree of turbulence; and a, a measure of the effect of the channel shape. Expressing Eq. 1 in terms of discharge by substituting Q/A for V,

$$S_e = \frac{Q^n}{A^n K R^a} \quad \ldots\ldots [2]$$

In order to analyze the problems of unsteady, non-uniform flow in open channels, it is necessary to solve Eq. 2 simultaneously with the varied-flow equation,

$$\frac{dE}{dx} = -S_e = S_s + \frac{\partial(\alpha V^2)}{\partial x(2g)} + \frac{1}{g}\frac{\partial V}{\partial t} \quad \ldots [3]$$

in which slopes in the downward direction are considered positive. The terms on the right are known as the "surface-slope term," the "velocity-head term," and the "acceleration-head term." (See Harold A. Thomas, *The Hydraulics of Flood Movements in Rivers*, page 33.)

Experience in the application of Eq. 3 to the problems of stream gaging indicates that the velocity-head and acceleration-head terms generally are so small in relation to the other factors involved that they may be neglected, though it is essential to evaluate them for each gaging station and to determine their effects. This evaluation is a comparatively simple matter and will not be dealt with here.

In the practical problems of stream gaging, the slope of the water surface (S_s) at a given instant can only be evaluated by means of observations of fall through a reach, determined by simultaneous readings on gages set to the same datum and a sufficient distance apart so that the errors due to gage readings will be a negligible percentage of the total fall. The length of reach necessary to properly evaluate the fall may in some instances be so great that the effects of inaccuracies in gage datum,

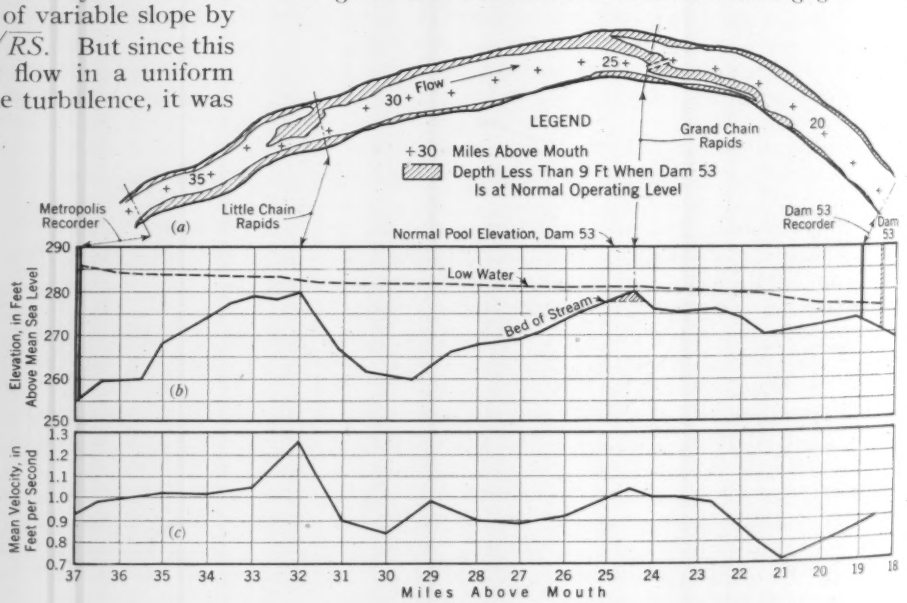

FIG. 1. OHIO RIVER IN VICINITY OF METROPOLIS, ILL.
(*a*) Plan; (*b*) Profile; (*c*) Approximate Mean Velocity for a Discharge of 50,000 Cu Ft per Sec at Pool Elevation 290.0

non-uniformity of the channel, entrance of tributaries, curvature of the water surface, and passage of a flood wave will make it difficult to correctly determine the fall. Because of these complications the relations indicated by Eq. 3 may best be expressed in general terms as a function of the observed slope of the water surface—that is, as

$$\frac{dE}{dx} = \phi\left(\frac{F}{L}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots [4]$$

in which F is the fall in the reach and L is the length of the reach—the velocity-head and acceleration-head terms being accounted for in the functional relation. Combining Eqs. 2 and 4,

$$\frac{Q^n}{A^n K R^a} = \phi\left(\frac{F}{L}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots [5]$$

Of course L is constant, and at any given stage A, R, and K are also constant (K may vary slightly with slope). Hence two discharges, Q_1 and Q_2, at the same stage may be related from Eq. 5,

$$\frac{Q_1}{Q_2} = \psi\left(\frac{F_1}{F_2}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots [6]$$

the exponent n having been included as a part of the function of fall. The function ψ is exponential, and may be most complex, because of the effects of non-uniformities in the channel reach, the fact that the gages may not be set to the same datum, the location of the gages with respect to stream regimen, the effects of bridge piers or intake drawdown, and many other factors. However, by graphical methods of analysis of Eq. 6 the relations between slope, stage, and discharge at a gaging station may be determined.

The following is a brief outline of the steps necessary in the graphical solution of Eq. 6. Plot the measured discharges as abscissae against the gage heights for the upper gage of the fall reach as ordinates, and write beside each plotted point the corresponding fall. Examination of these points will indicate that for some particular value of fall the discharge curve may be drawn by inspection throughout most of the range in stage. This fall is designated as the "normal" fall, F_n, and the discharge from the curve for the normal fall is designated as the "normal" discharge, Q_n. Other falls and the corresponding measured discharges are indicated by F_m and Q_m, respectively.

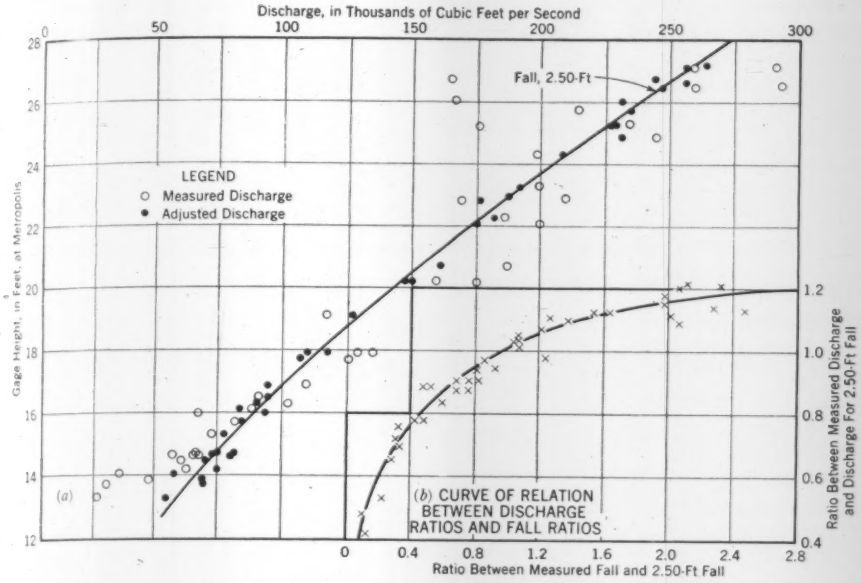

FIG. 2. SLOPE-STAGE-DISCHARGE RELATIONS, OHIO RIVER AT METROPOLIS, ILL. (*a*) Unadjusted and Adjusted Discharge Measurements; (*b*) Relation between Discharge Ratios and Fall Ratios

Substituting these symbols for F_1, and Q_1, and F_2, and Q_2, Eq. 6 becomes

$$\frac{Q_m}{Q_n} = \psi\left(\frac{F_m}{F_n}\right) \quad \ldots\ldots\ldots\ldots [7]$$

From the curve of stage-discharge relation for normal fall, Q_n for any stage can be read and substituted in Eq. 7. Then, substituting the Q_m and the F_m from an actual discharge measurement made at the same stage, the discharge ratio $\frac{Q_m}{Q_n}$ and the fall ratio $\frac{F_m}{F_n}$ can be computed. This procedure is followed for each discharge measurement, the results being plotted with discharge ratios as ordinates and fall ratios as abscissae. The curve which these points define is the graphical representation of Eq. 7.

Each discharge measurement is then adjusted to the corresponding normal discharge by dividing by the discharge ratio as obtained from the curve just plotted. It may be found after the adjusted discharges are plotted that the curve of stage-discharge relation for the normal fall should be corrected slightly. In such case, by repeating the steps outlined (using the corrected curve of stage-discharge relation), new discharge ratios can be computed and plotted and the discharge-ratio–fall-ratio curve corrected if necessary.

To illustrate the practical application of these methods, a Geological Survey gaging station at Metropolis, Ill., on the lower reaches of the Ohio River, may be taken as an example. The upper one of the two gages involved is at Metropolis, Ill., and the lower one is about 18 miles downstream, a short distance above Dam 53. The backwater at the Metropolis station is created primarily by the operation of Dam 53, a navigation structure; and also, at times, by high stages on the Mississippi River and by the return of over-bank flow following floods of unusual magnitude.

Between the gages there are two series of rapids known as the Grand Chain and the Little Chain. These rapids form marked breaks in the river profile and may, at times of medium or low discharge when Dam 53 is not operating, act as almost completely controlling sections for the upper gage. Figure 1 illustrates the plan and profile of the reach. It will be noted that, at times when the pool above Dam 53 is at normal operating level, the water over practically all of the river channel in the vicinity of these rapids is less than 9 ft deep, as represented by the hatched areas in Fig. 1(*a*). Figure 1(*b*) illustrates the profile of approximately the lower third of the cross-section of the stream bed, and the marked effect of the rapids on that profile may be noted. In Fig. 1(*c*) is shown the curve of average velocity through the reach for a discharge of 50,000 cu ft per sec, with Dam 53 at normal operating level. This illustrates the effects of rapids in a natural stream on the average velocity. Rapids are sections of reduced cross-sectional area, the reduction being created by shoals in the river bed, nar-

rowing of the stream width, or both, and as a result of this reduced area the velocity at the constricted section is greater than at sections in adjacent reaches. The increase in average velocity at each section of reduced area is clearly shown.

Figure 2 illustrates the curves of relation, developed in the manner described. One feature of the discharge-ratio–fall-ratio curve in this particular case is worthy of note: the curve becomes asymptotic to a horizontal line corresponding to a discharge ratio of 1.2 at a fall ratio of approximately 2.0. This indicates that for all falls above 5.0 ft (fall ratio 2.0) the rapids become the controlling sections for the upper gage, and no backwater effects can pass through them.

Having established the relation between slope (or fall), stage, and discharge in a channel reach, it is a simple matter to convert these relations into any of the several curves that may be found convenient in the determination of discharge. For example, it may be convenient to have a graphical representation of curves of equal gage heights at the lower gage plotted against the gage heights at the upper gage as ordinates and the discharge as abscissae. Such curves for the Ohio River at Metropolis, Ill., are shown in Fig. 3. It will be noted that, as the discharge and stage increase, these curves become asymptotic to a single curve, which is the stage-discharge relation for the upper gage when backwater is not present. As the discharge and stage decrease, each curve approaches the horizontal and at zero discharge intercepts the vertical axis at the stage it represents.

Another method of graphically representing the slope-stage-discharge relations is by means of curves of equal fall plotted against the stage at the upper gage and the discharge. These curves also are illustrated in Fig. 3. Curves of this type have the advantage over curves of equal stage in that discharges for conditions of low fall may be read from them with greater accuracy.

No attempt has been made to discuss the effects of the slope-stage-discharge relation of changing stage in connection with changing discharge. This is also a problem of variable slope, and methods have been devised by the Geological Survey for its solution, but space does not permit outlining them here.

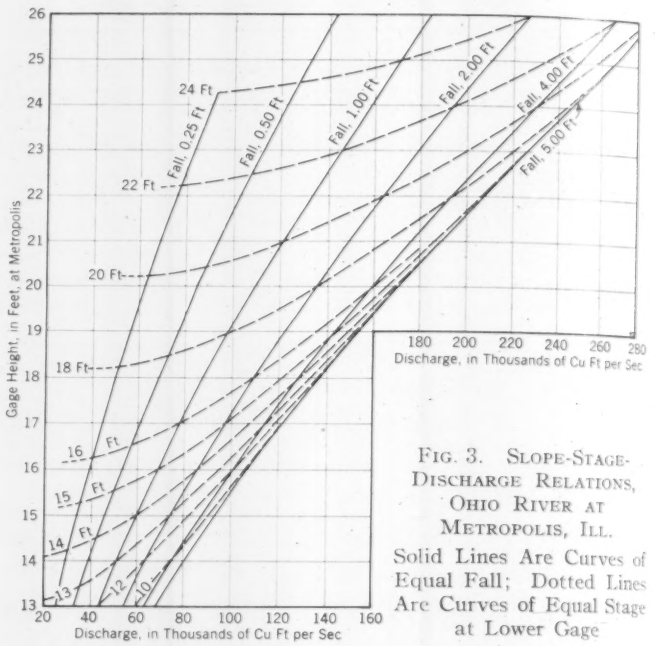

FIG. 3. SLOPE-STAGE-DISCHARGE RELATIONS, OHIO RIVER AT METROPOLIS, ILL. Solid Lines Are Curves of Equal Fall; Dotted Lines Are Curves of Equal Stage at Lower Gage

ENGINEERS' NOTEBOOK

This department, designed to contain ingenious suggestions and practical data from engineers both young and old, should prove helpful in the solution of many troublesome problems. Reprints of the complete department, 8½ by 11 in., suitable for binding in loose-leaf style, are available each month at 15 cents a copy.

Determining the Mean of Repetition Angles

By FLOYD W. HOUGH, M. AM. SOC. C.E.
PITTSBURGH, PA.

THERE is a simpler method of determining the mean value of an angle measured by repetitions than that generally shown in textbooks and apparently in common use. In this simplified method, the mean of the two arcs passed over is obtained directly by treating the D_0 and R_6 as two "initials," in the same manner as that used in the direction method of angle observations.

A "Form of Field Notes, Repetition Method, for Two-Vernier Theodolite" is presented in Table 11 of *Technical Procedure for City Surveys* (Am. Soc. C.E., Manuals of Engineering Practice, No. 10). The first and fifth angles of that table are repeated here as Table I. It is noted that in this table, two "arcs passed over" have been obtained in each case, each of these arcs has been divided separately by 6, the two results have been added, and one-

TABLE I. FIELD NOTES, "STANDARD" METHOD

Station Sighted	Telescope Direct Reversed	No. of Repetitions	Circle Reading	Verniers* A	Verniers* B	Verniers* Mean	Arc Passed Over	Angle	Mean
Lynn	D	0	50° 00′	55″	00″	57.5″			
Homewood..	D-R	6	160° 08′	40″	30″	35″	110° 08′ 37″.5	78° 21′ 26″.2	26″.4
Set No. 3...	R-D	6	50° 00′	55″	55″	55″	249° 51′ 20″.0	281° 38′ 33″.3	
								−0″.5	
Homeo.....	D	0	50° 00′	00″	00″	00″			
E. End.....	D-R	6	357° 25′	35″	25″	30″	307° 25′ 30″	51° 14′ 15″.0	13″.6
Set No. 3...	R-D	6	50° 00′	15″	20″	17″.5	52° 34′ 47″.5	308° 45′ 47″.9	
								+2″.9	

* Barred numerals in seconds indicate a reading one minute less than recorded.

half th small difference between this sum and 360 deg has been applied, with its proper sign, to one-sixth of the arc passed over on the direct measurement.

If we determine, purely by inspection of the vernier-means column, the midway point on the circle between the D_0 and the return R_6, which for want of a better term we may call the "mean initial" (I_m), we have but to subtract this value from D_6 and to divide the result by 6 to get the correct mean angle as shown in the revised form (Table II). A third angle is listed to include a condition where D_0 and R_6 are on opposite sides of zero.

TABLE II. FIELD NOTES, "SIMPLIFIED" METHOD

Station Sighted	Tel. Rep.	Circle Reading	Verniers A	Verniers B	Verniers Mean	I_m	$D_6 - I_m$	Mean Angle $((D_6 - I_m) \div 6)$
Lynn	D_0	50° 00′			55″.00 57″.5			
Homewood	D_1	128° 21′			20″			
Set No. 3	D_6	160° 08′			40″.30 35″.0	56.2	110° 08′ 38″.8	78° 21′ 26″.5
	R_6	50° 00′			55″.55 55″.0			
Homeo	D_0	50° 00′			00″.00 00″.0			
E. End	D_1	101° 14′			10″			
Set No. 3	D_6	357° 25′			35″.25 30″.0	08.8	307° 25′ 21″.2	51° 14′ 13″.5
	R_6	50° 00′			15″.20 17″.5			
Bat	D_0	00° 00′			10″.10 10″.0			
Kow	D_1	27° 34′			10″			
Set No. 3	D_6	165° 26′			20″.30 25″.0	02.5	165° 26′ 22″.5	27° 34′ 23″.8
	R_6	00° 00′			50″.00 55″.0			

The values here shown by the shorter method (Table II) are correct to the nearest 0.1″, while those in Table I are not, due to the dropping of decimals in the larger number of operations. The subtraction, $D_6 - I_m$, is so simple that it may be done mentally by inspection.

Incidentally, the reading D_1 on Vernier A only (which is omitted in Table I) should always be recorded. It furnishes a check on the degrees and minutes of the final angle and facilitates the work, especially when D_6 exceeds 360°.

The simpler method, called to my attention several years ago and here recommended, is theoretically correct and gives the exact mean angle. It is less laborious and is not nearly so susceptible to error as the longer method for the reasons that there are fewer operations and that it does not involve the application of a small correction at the end with its proper algebraic sign.

Runoff Distribution Graphs from Precipitation Occurring in More Than One Time Unit

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THE method used by the Muskingum Watershed Conservancy District in finding sets of coefficients or graphs for runoff distribution, for a rain occurring in more than one time interval, may be of interest. This method seems simpler than using least squares, graphical analysis, or other published methods. Means of determining the portion of each rain that should be considered as runoff, and the manner of plotting a distribution graph from its coefficients are also discussed, since they are so closely allied.

When distribution graph coefficients are to be determined, the first factor that must be considered is the time unit. There are several things that influence this, one of which is the precipitation records. If these are all in the form of daily totals, the minimum time interval for the distribution graph is necessarily 24 hours. However, on or near most watersheds there is at least one recording rainfall gage, and notation of the time of showers by observers is sometimes available also, so that precipitation can in many cases be apportioned to intervals of 12 hours or less. Another factor in selecting the interval is the time it takes the stream being studied to reach its crest. The interval should always be shorter than the time to the crest, and preferably half this time or less, in order to define the shape of the hydrograph.

Actual computation of the distribution coefficients is by a systematic trial-and-error method. After the time interval has been selected, the average precipitation for each interval is listed. The total amount of surface runoff is estimated from the hydrograph, by subtracting a reasonable base flow, and assigned to the various intervals of rain. This may be done by whatever method the user prefers. The possible methods are (1) to use the same percentage of precipitation for all intervals, (2) to subtract a constant from all precipitation values, (3) to use infiltration rates (if there are enough data on precipitation intensities), or (4) to utilize a mixture of these methods tempered strongly by judgment based on experience and knowledge of the attending conditions. The use of a percentage is the most convenient and seems to have some validity; for even when the average intensity of a rain is less than the average infiltration rate there is runoff, because of the non-uniformity of both the precipitation and the infiltration rate over the drainage basin—that is, there is some runoff from practically every rain. Subtracting a constant is a corruption of the infiltration rate method, based on the assumption that every rain continues uniformly throughout the period in which it occurs.

The method used by the District as a "starter" is a combination of these two. First, a constant loss, large enough to account for about half the total loss, is subtracted from the precipitation of each period; then a percentage is applied to the remainders to make the total equal the surface runoff. If it is thought that these results do not fit the conditions, judgment is applied freely in their modification. Usually the data are not sufficient for use of the infiltration rate method, although it appears to be best.

After the inches of runoff for each period of rain have been determined, they are converted to cubic feet per second per time unit. A distribution graph for the drainage area is estimated and used on all of them except the largest. The discharges from the smaller runoff amounts and the base flow are all subtracted from the hydrograph. The residual is then considered to be the distribution of discharge from the largest runoff amount, and percentage coefficients are easily computed from this. The new set of coefficients usually will be more nearly correct than the original estimated set. This is true be-

cause an error made in assuming the trial coefficients does not cause as large a percentage of error in the result, since the trial coefficients are applied to smaller runoff amounts than that from which the new coefficients are found.

Coefficients from this distribution are compared with the original estimated set, and if there is little difference between them they are used as found. However, if there is a material difference between the estimated and the computed coefficients, the procedure must be repeated to prevent large errors in the trial set from being reflected in the final distribution graph. In this repetition, the trial distribution coefficients used are an average of the two sets, weighted according to the total runoff to which each applied during the time over which the largest runoff amount is distributed. After the second trial the computed and trial sets will usually agree well enough.

A sample computation of a set of distribution coefficients for the Tuscarawas River near Dover, Ohio, for the storm of August 6–7, 1935, is shown in Table I. The drainage area of this stream is 1,398 sq miles. The precipitation used is, for convenience, a straight average of the Akron, Canton, Dover, Millport, and Wooster values. The Akron and Wooster records were available in hourly form. These, with notes on beginning and ending times of showers at the other stations, where readings were made only daily, made it possible to apportion precipitation to half-day intervals (Col. 3). Inspection of the preceding and succeeding parts of the hydrograph gave 600 cu ft per sec as a reasonable base flow (Col. 7). This was subtracted from the hydrograph of actual flow (Col. 6), to find the surface runoff (Col. 8).

The total surface runoff for this flood was 226,600 half-day second feet, or 3.01 in. over the drainage area. This quantity was accordingly apportioned (Cols. 4 and 5) to the periods when there was rain, chiefly on the basis of judgment. Each of these runoff amounts, except the largest (between the eighteenth hour of August 6 andthe sixth hour of August 7), was then distributed to successive periods, starting with the one in which it occurred.

The trial coefficients used for this distribution (heads of Cols. 9 to 22) are the average of several previous determinations. The sums (Col. 23) of the distributed runoffs (Cols. 9 to 22) are approximately the runoff as it would have occurred without the rain of the night of August 6–7, and prior to that time should agree with the surface runoff (Col. 8). The residuals (Col. 24), obtained by subtracting Col. 23 from Col. 8, represent the runoff from that rain. Values of the residuals before the start of that rain should be zero, and, as they stand, are the result of discrepancies in observation or technique. However, the largest residual is less than 1 per cent of the August 6–7 runoff, which is within the accuracy of the method.

As the final step, the items in Col. 24, beginning at Hour 18 on August 6 and representing surface runoff from the August 6–7 rainfall, were each divided by the runoff amount (125.0 from Col. 5) for that rain, to find the per cent (Col. 25) that appeared as runoff in successive periods. This is the direct distribution graph of this flood.

In this instance three coefficients vary by two units from the trial values. This was considered close enough

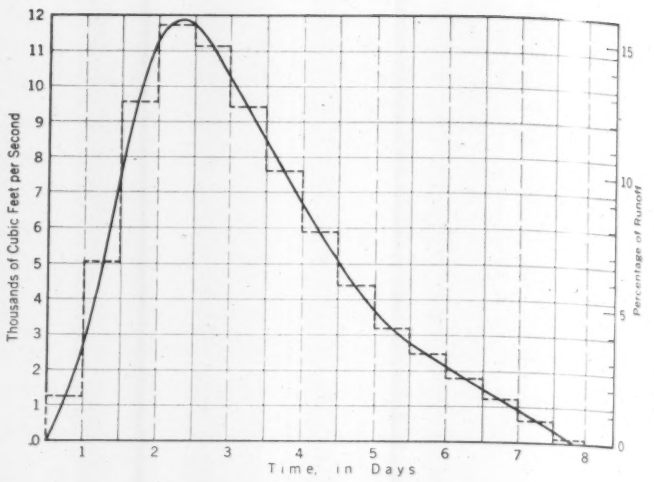

FIG 1. UNIT GRAPH FOR TUSCARAWAS RIVER, NEAR DOVER, OHIO, FOR FLOW RESULTING FROM 1 IN. OF RUNOFF (RAIN OCCURRING IN ONE HALF DAY)

TABLE I. DETERMINATION OF HALF-DAY COEFFICIENTS FOR RUNOFF DISTRIBUTION OF TUSCARAWAS RIVER NEAR DOVER, OHIO (AUGUST 1935)

(1) Day	(2) Hour	(3) Precipitation, In.	(4) Runoff Amounts, In.	(5) 1,000 ½ dsf*	(6) Aver. Disch. 1,000 cfs	(7) Base Flow 1,000 cfs	(8) Col. 6–7	(9) 2	(10) 7	(11) 13	(12) 16	(13) 15	(14) 13	(15) 10	(16) 7	(17) 6	(18) 4	(19) 3	(20) 2	(21) 1	(22) 1	(23) Sum	(24) Residual	(25) Computed Coeff. %
								Trial Coefficients, Per Cent																
2	6	0.88	0.16	12.0	0.9	0.6	0.3	0.2	...	...	...	...	...	...	...	...	...	...	...	...	0.2	0.1	...
	18	1.48	0.52	39.3	1.3	0.6	0.7	0.8	0.8	...	...	...	...	...	...	...	...	...	...	...	1.6	−0.9	...
3	6	..	..	..	5.3	0.6	4.7	....	2.8	1.6	...	...	...	...	...	...	...	...	...	...	4.4	0.3	...
	18	0.48	0.16	12.0	8.0	0.6	7.4	0.2	...	5.1	1.9	...	...	...	...	...	...	...	...	...	7.2	0.2	...
4	6	..	..	..	8.6	0.6	8.0	...	0.8	...	6.3	1.8	...	...	...	...	...	...	...	...	8.9	−0.9	...
	18	..	..	..	9.4	0.6	8.8	...	...	1.6	...	5.9	1.6	...	...	...	...	...	...	...	9.1	−0.3	...
5	6	..	..	..	9.2	0.6	8.6	...	...	...	1.9	...	5.1	1.2	...	...	...	...	...	...	8.2	0.4	...
	18	..	..	..	7.8	0.6	7.2	...	...	...	...	1.8	...	3.9	0.9	...	...	...	...	...	6.6	0.6	...
6	6	0.45	0.16	12.0	6.0	0.6	5.4	0.2	...	...	...	...	1.6	...	2.8	0.7	...	...	...	...	5.3	0.1	...
	18	2.46	1.66	125.0	6.2	0.6	5.6	...	0.8	...	...	...	...	1.2	...	2.3	0.5	...	...	...	4.8	0.8	1
7	6	..	..	..	13.5	0.6	12.9	...	...	1.6	...	...	...	...	0.9	...	1.6	0.4	...	...	4.5	8.4	7
	18	..	..	..	21.3	0.6	20.7	...	...	...	1.9	...	...	...	...	0.7	...	1.2	0.2	...	4.0	16.7	13
8	6	0.25	0.06	4.5	21.7	0.6	21.1	0.1	...	...	...	1.8	...	...	...	...	0.5	...	0.8	0.2	3.4	17.7	14
	18	..	..	..	19.6	0.6	19.0	...	0.3	...	...	...	1.6	...	...	...	...	0.4	...	0.5	0.1	2.9	16.1	13
9	6	..	..	..	17.2	0.6	16.6	...	...	0.6	...	...	...	1.2	...	...	...	...	0.2	...	0.3	2.3	14.3	11
	18	..	..	..	14.9	0.6	14.3	...	...	...	0.7	...	...	...	0.9	...	...	...	...	0.2	...	1.8	12.5	10
10	6	0.39	0.24	18.0	12.6	0.6	12.0	0.3	...	...	...	0.7	...	...	...	0.7	...	...	...	...	0.1	1.8	10.2	8
	18	..	..	..	11.3	0.6	10.7	...	1.2	...	...	...	0.6	...	...	...	0.5	...	...	...	...	2.3	8.4	7
11	6	..	..	..	10.5	0.6	9.9	...	...	2.4	...	...	...	0.4	...	...	...	0.4	...	...	...	3.2	6.7	5
	18	..	..	..	9.1	0.6	8.5	...	...	...	2.9	...	...	...	0.3	...	...	...	0.2	...	...	3.6	5.1	4
12	6	..	..	..	7.3	0.6	6.7	...	...	...	...	2.7	...	...	...	0.3	...	...	...	0.2	...	3.2	3.5	3
	18	..	..	..	5.4	0.6	4.8	...	...	...	...	...	2.4	...	...	...	0.2	...	...	...	0.1	2.7	2.1	2
13	6	0.24	0.05	3.8	4.0	0.6	3.4	0.1	...	...	...	...	...	1.8	...	...	...	0.1	...	...	...	2.0	1.4	1
	18	..	..	..	3.1	0.6	2.5	...	0.3	...	...	...	...	...	1.3	...	...	...	0.1	...	...	1.7	0.8	1
14	6	..	..	..	2.7	0.6	2.1	...	...	0.5	...	...	...	...	...	1.1	...	...	...	0.1	...	1.7	0.4	0
	18																							

* 1 half-day second foot = inches × 75,200 for this drainage area (1,398 sq miles).

agreement. If it had been necessary to recompute this set of coefficients, the new trial coefficients would have been a weighted average of the set used before and those just found. The weight in determining the average would be:

(*a*) For the computed coefficients, the sum of Col. 24 from Hour 18 on August 6 to Hour 6 on August 14;

(*b*) For the old trial coefficients, the difference of the sum found in (*a*) and the sum of Col. 8 for the same periods.

If the coefficients are plotted to form a distribution graph, a block hydrograph should be used, not just a series of lines connecting them. After the block hydrograph has been constructed, a smooth curve may be drawn in such manner that the area for each period is not changed. The graph shown in Fig. 1 has been prepared in this way. It is based on the analysis of 14 storms, all of which gave coefficients differing by not more than 2 per cent from the corresponding coefficients in any other set.

No one storm will give distribution coefficients suitable to all others. This is partly because of variation in intensity and areal distribution of precipitation within individual periods. The shape of a distribution graph is also affected by duration of the storm and amount of runoff. The influence of channel or valley storage is assumed, when applying distribution graphs, to be proportional to the runoff amounts of the various periods. However, those things which change distribution graphs do so through their effect on storage. Caution therefore should be exercised when applying a unit graph to a storm of such characteristics that storage may be materially different from what it was when the coefficients were found. Several storms should be analyzed. When using distribution coefficients they should be selected with the storm characteristics in mind.

Deflection of Plate Girders of Variable Depth

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A SIMPLY supported plate girder having a curved bottom flange and a half-length equal to l is shown in Fig. 1. The x-axis is taken along the center of gravity of the top flange, the origin being at the mid-point. The beam is symmetrical about the center line, and the depth, or d-axis, is taken positive downward.

The depth between the centers of gravity of the flanges of the beam at any point, x, is given by the equation

$$d = d_c\sqrt{1 - \frac{ax^2}{l^2}} \quad \ldots\ldots\ldots\ldots\ldots [1]$$

where d_c is the depth at the center, and a is a known constant.

The moment of inertia, I, at any point, x, is then approximately

$$I = 2A\left(\frac{d}{2}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots [2]$$

where A is the area of one flange consisting of angles, cover plates if any, and sometimes a portion of the web plate. Top and bottom flanges are assumed to have equal areas.

Substituting Eq. 1 in Eq. 2 gives

$$I = \frac{Ad_c^2}{2}\left(1 - \frac{ax^2}{l^2}\right) \quad \ldots\ldots\ldots\ldots\ldots [3]$$

Since the moment of inertia at the center, I_c, is $\frac{Ad_c^2}{2}$, Eq. 3 becomes

$$I = I_c\left(1 - \frac{ax^2}{l^2}\right) \quad \ldots\ldots\ldots\ldots\ldots [4]$$

which may be rewritten as

$$I = \frac{I_c}{1 + \frac{ax^2}{l^2} + \frac{a^2x^4}{l^4} + \frac{a^3x^6}{l^6} + \ldots} \quad \ldots\ldots\ldots [5]$$

The series in the denominator is rapidly convergent for values of a less than unity.

The approximate relationship between the curvature and bending moment in a beam is

$$\frac{d^2y}{dx^2} = -\frac{M}{EI} \quad \ldots\ldots\ldots\ldots\ldots [6]$$

Substituting Eq. 5 in Eq. 6 yields

$$\frac{d^2y}{dx^2} = -\frac{M}{EI_c}\left(1 + \frac{ax^2}{l^2} + \frac{a^2x^4}{l^4} + \frac{a^3x^6}{l^6} + \ldots\right) \quad [7]$$

Since M is a function of x only, substitution for the bending moment may be made in Eq. 7 and the value of y obtained by two successive integrations. This will be illustrated by two examples.

Example 1, Concentrated Load at Center. Figure 2(*a*) shows the beam loaded by a concentrated force at the center. The bending moment at point x is

$$M = \frac{Pl}{2}\left(1 - \frac{x}{l}\right) \quad \ldots\ldots\ldots\ldots\ldots [8]$$

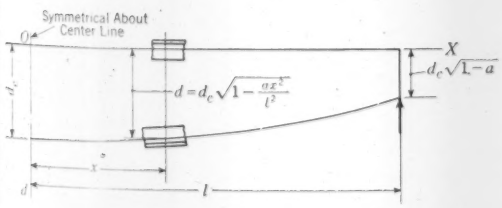

FIG. 1. PLATE GIRDER OF VARIABLE DEPTH

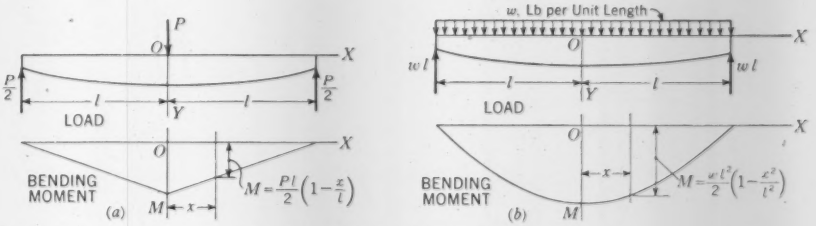

FIG. 2. ILLUSTRATIVE EXAMPLES
(*a*) Concentrated Load at Center, (*b*) Uniformly Distributed Load

Substituting in Eq. 7, removing parentheses, and integrating twice gives

$$y = -\frac{Pl}{2EI_c}\left(\frac{x^2}{2} - \frac{x^3}{2\cdot 3l} + \frac{ax^4}{3\cdot 4l^2} - \frac{ax^5}{4\cdot 5l^3} + \frac{a^2x^6}{5\cdot 6l^4} - \frac{a^2x^7}{6\cdot 7l^5} + \cdots\right) + C_1x + C_2 \quad [9]$$

The constant of integration C_1 is evaluated by considering the slope of the beam at the center. For $x = 0$, $\frac{dy}{dx}$ is also zero; hence $C_1 = 0$. At the end of the beam, $x = l$ and $y = 0$. Hence

$$C_2 = \frac{Pl^3}{EI_c}\left(\frac{1}{1\cdot 2\cdot 3} + \frac{a}{3\cdot 4\cdot 5} + \frac{a^2}{5\cdot 6\cdot 7} + \frac{a^3}{7\cdot 8\cdot 9} + \cdots\right) \quad [10]$$

The deflection curve of the beam is completely determined by Eqs. 9 and 10. The maximum value of the deflection occurs at the center, $x = 0$, and becomes

$$y_{max} = C_2 \quad [11]$$

For comparing the deflection with that of a beam of uniform cross-section, consider a girder whose depth at the support is one-half as great as at the center. For such a beam, a would be taken as 0.75.

Substitution in Eqs. 10 and 11 gives

$$y_{max} = 0.183\frac{Pl^3}{EI_c} \quad [12]$$

This may be written in terms of the full span, l_1, where $l = \frac{l_1}{2}$ as

$$y_{max} = \frac{1.10Pl_1^3}{48EI_c} \quad [13]$$

Such a beam is thus seen to have 10 per cent greater deflection at the center than one whose moment of inertia is I_c throughout.

Example 2, Beam with Uniform Load. As shown in Fig. 2(*b*), the bending moment is

$$M = \frac{wl^2}{2}\left(1 - \frac{x^2}{l^2}\right) \quad [14]$$

Substituting in Eq. 7, removing parentheses, and integrating twice yields

$$y = -\frac{wl^2}{2EI_c}\left[\frac{x^2}{2} - (1-a)x^2\left(\frac{x^2}{3\cdot 4l^2} + \frac{ax^4}{5\cdot 6l^4} + \frac{a^2x^6}{7\cdot 8l^6} + \frac{a^3x^8}{9\cdot 10l^8} + \cdots\right)\right] + C_1x + C_2 \quad [15]$$

By consideration of the same boundary conditions as in Example 1, the constant of integration, C_1, is found to be equal to zero, and C_2 is given by

$$C_2 = \frac{wl^4}{2EI_c}\left[\frac{1}{2} - (1-a)\left(\frac{1}{3\cdot 4} + \frac{a}{5\cdot 6} + \frac{a^2}{7\cdot 8} + \frac{a^3}{9\cdot 10} + \cdots\right)\right] \quad [16]$$

The maximum deflection occurs at the center, $x = 0$, and becomes

$$y_{max} = C_2 \quad [17]$$

For comparison with a beam of uniform cross-section, again consider a girder whose depth at the end is one-half that at the center ($a = 0.75$). The maximum deflection is found to be

$$y_{max} = 0.234\frac{wl^4}{EI_c} \quad [18]$$

This may be written in terms of the full span, l_1, as

$$y_{max} = \frac{5.62\ wl_1^4}{384\ EI_c} \quad [19]$$

This beam is thus seen to have a deflection at the center 12.4 per cent greater than one whose moment of inertia is constant and equal to I_c.

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

How Can a Building Code Be Enforced?

TO THE EDITOR: The article, "A Modern Building Code—Arrangement and Organization," by D. S. Laidlaw, in the June issue, is unique in that it treats of the classification and arrangement of the content of a building code. Mr. Laidlaw's observations and suggestions are well worth consideration. A similar article on how a building code can be enforced would be welcome.

This is of prime importance—who shall decide whether the design of a proposed building meets the requirements of the code which governs its construction? Two reminiscences come to mind. In the days of iron highway bridges, the bids for bridges in a certain county in Ohio were referred to the county surveyor to decide whether the design complied with the specifications. I once asked this individual how he passed on compression members. He replied, "I divide the strains given on the strain sheet by 10,000 to find out what section is needed."

Again, a few years ago a certain city required that its building code be revised. I met the committee in charge of the revision. In the draft of a proposed code they had specified a live load on office floors of 80 lb per sq ft. To my suggestion of 50 lb, one member said that he had seen an office floor loaded with brick and that the load was certainly 80 lb per sq ft. Another member stated that he fully agreed 50 lb was sufficient. At the same time, he continued, if they called for 70 or 80 lb they would probably get 50 or 60 lb, but if they specified 50 lb no one knew what they would get. This member's opinion favorably impressed his fellows, and 80 lb was recommended. However, the present code calls for 70 lb per sq ft, with varying reductions on columns from floors of tier buildings.

ROBINS FLEMING
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Field Control of Compacted Earth Fill

DEAR SIR: I am very much interested in E. A. Abdun-Nur's discussion in the June issue of my article, "Field Control of Compacted Earth Fill," in the April issue. As that article was an abridgment of the much longer original, which is in the Engineering Societies Library, I naturally could not cover in detail many of the points brought up by Mr. Abdun-Nur.

As a justification for basing density requirements on laboratory tests and checking these results by testing undisturbed samples from the fill, I specified in the original paper that the testing should be done in one of the most advanced laboratories in the country and mentioned several—Harvard, Princeton, U. S. Engineer Corps, Massachusetts Institute of Technology, Case, and others. The correlation between the test results from those laboratories and field conditions is scattered throughout technical literature, and the unification of testing technique is being carried on very rapidly in the Soil Mechanics and Foundations Division of the Society. In addition, my original paper stated that the test results should be interpreted in the light of commonsense and practical experience, but in so stating I was not making excuses for the present testing equipment and technique as used and practiced at the above-mentioned laboratories. With that in mind, I believe we are well justified in basing density requirements on laboratory tests. But the point raised by Mr. Abdun-Nur is well taken; testing should not be carried on before checking on the very latest developments in this field.

In using the oil method of testing density, the viscosity of the oil should be determined by experiment; I selected a viscosity which lowered the percentage of error to a point where I could not measure it with the field equipment I was using. The sand method used by Mr. Abdun-Nur is an excellent direct method of measuring density, as is also the method of using a knife-edge pipe spoon to extract an undisturbed sample of exactly 0.1 cu ft. Both these methods are discussed in the original paper, with the reasons for preferring the oil method.

Mr. Abdun-Nur's comments on my method of drying samples over an open coal or charcoal fire are true, but numerous check samples dried in the electric oven never showed an error greater than one-half of 1 per cent of the dry weight. As I had charts showing the relationship between density and number of passes with the rollers at different moisture contents, the error in taking moisture content by this quick method was too small to affect the results. I prefer this quick but slightly inaccurate method to the alternative of drying a sample in an electric or gas oven which may take from three-quarters of an hour to a whole day, thus delaying orders to the constructor to change the number of passes.

As Mr. Abdun-Nur points out, the type of material may allow the use of the penetration needle for determining moisture content. However, my experience, based on exhaustive tests over a period of two years, has been that the direct method of drying samples over a coal or charcoal fire gave more accurate results. It is interesting to note in this connection that Mr. Abdun-Nur used a direct method of determining density and did not use the needle for density control.

HOWARD F. PECKWORTH, M. Am. Soc. C.E.
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Santee-Cooper Project
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Problems in Valuation

TO THE EDITOR: In the article by E. De V. Tompkins entitled "Value of 'Light, Air, and Access' Easements," which appeared in the June issue of CIVIL ENGINEERING, four problems in valuation were presented. I wish to submit a theory of valuation which yields different results from those of the author in Problems 1, 3, and 4, but gives the same answer he obtained in Problem 2.

Problem 1 assumed that, after 20 years of existence, an elevated railroad property is condemned. The court finds that its physical property has only junk value and that its franchise, though still being operated, is worthless. Its "light, air, and access" property easement rights were acquired for a 50-year period, for the sum of $94,571. What is their value now, when condemned? It was assumed that 6 per cent would be a proper interest rate.

The original payment of $94,571 would have provided a series of 50 year-end payments of $5,996 each, and at the end of the 50 years the original sum would have been completely paid out. Since the plan was terminated at the end of only 20 years, it is my theory that the amount of money due the company at this time is the amount left after 20 year-end payments have been deducted from the original sum at the chosen interest rate. This amount for Problem 1 would be $82,704.

Applying this method to the Sixth Avenue Elevated case, the solution is as follows. The original easement cost "in perpetuity" was $4,346,000. In other words, this sum was assumed to be sufficient to provide year-end payments at 6 per cent equal to the loss in annual income from the abutting property forever. Therefore, at the end of any year the original sum of $4,346,000 would still be left to the credit of the elevated company and would constitute the amount of money which should be paid to it at the date of condemnation. The only justification, as I see it, for paying the full amount awarded to the Sixth Avenue Elevated Company lies in the phrase "in perpetuity." Had a specific term for the life of the easement been fixed at the time of its purchase, a considerable saving could have been effected at the date of condemnation. The solution to this vexing problem of valuation, therefore, appears to be not to allow easements to be purchased in perpetuity.

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Training Young Engineers in the Fundamentals

TO THE EDITOR: Professor Finch, in his article entitled "Practicing Engineers as Educators," in the June issue of CIVIL ENGINEERING, raises an important question—that of providing real apprenticeships for graduates of schools of engineering, comparable to the internships available to graduates of medical schools. The problem has been solved for medical students rather satisfactorily. It will be more difficult to do the same for the young engineer. How is it to be done? Hospitals are not in competition as are engineering firms or industrial establishments in which engineering plays an important part. Only the largest employers feel able to face a possible loss in teaching young engineers some of the principles of practice. Too often the young graduate is actually exploited, though there may be no intention on the part of the employer to exploit him. It is difficult for the employer to give the new employee more than he gets out of him immediately, with the result that he too often gives him a salary and nothing more. What is the solution?

Professor Finch's observation that in too many cases little or no attention is given to the degree an applicant for apprenticeship training may hold or to the reputation of the school from which he has graduated is a sad commentary on many of our schools of engineering. The practicing engineer would give more attention to these matters if they merited more attention—the very fact that he doesn't indicates that in the majority of cases they do not deserve much attention. The fault lies in our schools, not in our practicing engineers. I know of engineering-employers who definitely favor young graduates in physics for engineering jobs. It is felt, and experience has shown, that it is easier and produces better results to teach young physicists the practical side of engineering than it is to teach young engineers the physics they should have learned in school but were never required to. In fact if the fundamentals are not learned in school, they hardly ever are learned later.

What to do with the young engineer the first year or two he is out of school is an important problem and one that is worth discussing. What to do with him before he gets out is an equally important problem and one that is more easily solved. He must be trained in the fundamentals—physics, chemistry, mathematics. Four years in a college of liberal arts, with emphasis on physics, chemistry, and mathematics, followed by two years in a school of engineering staffed by a faculty grounded in the fundamentals (not always the case) are necessary, if present standards are to be raised appreciably.

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Formulating a Standard Building Code

TO THE EDITOR: Mr. Laidlaw's desire to formulate a standard set-up for building codes (see his article in the June issue of CIVIL ENGINEERING) can well be appreciated by anyone engaged in the industry. It is possible, however, that even the model code set-up is not necessarily the best, particularly after being subjected to numerous amendments and additions.

Without attempting to make any classification of the contents of building codes, we may say that they usually contain the following types of information: (1) code principles, which are accepted legal provisions that change very little or not at all; (2) minimum requirements—that is, standards of a certain type which are based on our conceptions of good practice and decency; and (3) rules and regulations, which are of a more specific nature and which, due to changes in conditions, are subject to continuous alteration.

The following items are not suggestions for any possible classification, but merely memoranda on some essentials that may deserve consideration.

Minimum Requirements: It is generally understood that by the very nature of the code, as a legal document, it cannot stipulate higher requirements than just enough to bring the conditions covered beyond the borderline from unsafe to safe territory. It would be better for all concerned if the code definitely stated which clauses are minimum requirements and then permitted no deviation from these requirements. Such a stipulation would eliminate a great deal of argument and give the administrators of the code some line of defense.

Principles and Rules: The building code proper should primarily state principles, all of which could be condensed into a very few pages. The principles should form the code part, to be amended only through legislative action. The code principles can be elaborated by rules issued from time to time and amended by a special technical advisory board. The rules can be argued, and dispensations can be given if satisfactory proof is submitted that the code principles are not violated.

Definitions: A great part of the building code is purely a matter of definitions. The best way out of this would be to elaborate greatly upon the chapter of definitions and include in it everything of a descriptive character.

Human Standards: Building codes also contain certain standards which have become accepted either through habits and conventions or through the unchangeable requirements of the human body. Such standards are expressed in the rules regarding the proportions of treads and risers on stairs, minimum number of risers, heights and construction of railings, and so forth. All these little things, which make a building right or, when not considered, make it a menace to the tenants and a source of damage suits to the owners, could and should be included with definite provisions that they constitute minimum requirements.

National Codes: As for the formulation of a national building code, it may be said that we already have a national, or even an international set of codes, which includes Canada. However, with the exception perhaps of the codes of the American Institute of Steel Construction, the American Concrete Institute, and the Canadian Standards Association, they are not applied in a sufficient degree to be of much value. All this work done by independent organizations is of little avail unless all building codes adopt these sub-codes and preferably add them as an appendix.

To sum up, let us (1) maintain the classification of types of buildings and improve it, if possible; (2) follow nationally accepted codes on structural standards, fire protection, and safety; (3) make uniform definitions and incorporate as many as possible under a combined index and glossary; and (4) include architectural (human) standards as minimum requirements.

SVEND PLUM, Assoc. M. Am. Soc. C.E.
Detroit, Mich.

The Maximum Probable Flood

TO THE EDITOR: Our article on "The Maximum Probable Flood and Its Relation to Spillway Capacity," which appeared in the January issue, has been discussed by Robert F. Ewald, in the April issue, and by C. Frank Johnson, in the May issue. Both commentators suggest that greater consideration should be given to probability methods. Mr. Ewald presents a chart showing the relative frequency of "great" storms in the eastern United States. The southern boundary of the area of no great storms is shaped not unlike the isohyetals on the rainfall frequency charts presented by David L. Yarnell (U. S. Department of Agriculture, Misc. Pub. No. 204). Figure 59 of this publication shows the 24-hour rainfall in inches, to be expected once in 100 years. Except for the "tongue" extending into Ohio on Mr. Ewald's chart, the 7-in. isohyetal in this figure agrees fairly well with the boundary of "no great storms." This phase of the problem was considered in preparing the charts of maximum probable rainfall, but the 100-year storm is, in many cases, no more than one-third to one-half the maximum that has occurred. The late Allen Hazen, in his book *Flood Flows*, referred to by Mr. Johnson, brings out vividly the effect of one large flood in the period of record on the probability calculations.

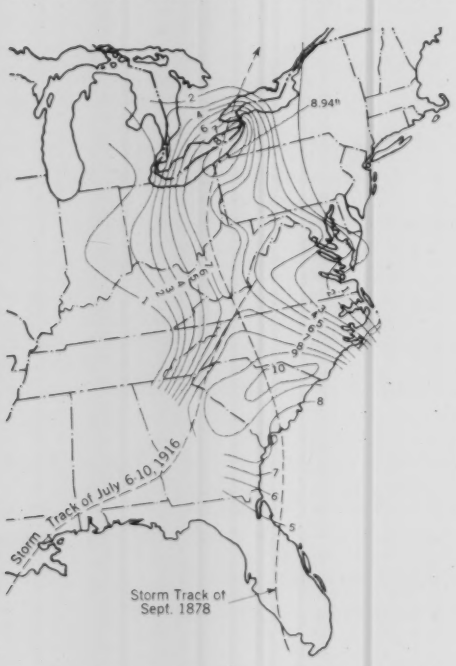

FIG. 1. ISOHYETAL MAP FOR STORM OF SEPTEMBER 10–13, 1878, AND TRACK OF JULY 6–10, 1916, STORM

Mr. Ewald takes particular exception to application of the 1,500 and 5,000-sq mile charts to the area about 200 miles wide along the western slopes of the Appalachians. This area is in a region subject to the passage of tropical hurricanes entering the country both from the South Atlantic and from the Gulf of Mexico. The period of record selected by Mr. Ewald—1892 to date—is insufficient to justify the conclusion that a great storm cannot occur over the shaded portions of his map. Many of the greatest floods for which there are definite data and which originated in West Virigina, eastern Kentucky, and Tennessee occurred prior to 1892. These floods resulted from the storms of September 1861, January 1862, March 1865, March 1867, and September 1878.

The first of these storms produced the greatest flood of record on the Lower Kanawha River at Charleston, W.Va.—5 ft above any other crest. The storm of 1862 produced the maximum flood of record on the Big Sandy River in Kentucky. The storm of March 1865 produced a large flood on the Cumberland River.

The storm of March 1867 was similar to that of July 6–10, 1916 (Technical Reports, Part V, Miami Conservancy District). This 1916 storm entered the country from the Gulf of Mexico and produced an average of 12 in. of rainfall over 2,500 sq miles in North Carolina. Records show that the storm of March 1867 followed the western side of the mountains (the region regarded as immune by Mr. Ewald) and produced the maximum flood of record in the Upper Tennessee River.

The storm of September 1878 (Fig. 1) was similar to those of September 1861 and July 14–16, 1916. This storm crossed the Appalachian divide and produced a rainfall which over a large area was reduced in intensity by only 2 in.—that is, from 10 to 8 in.—in 48 hours. Flood stages indicate that the precipitation of 1861 must have been greater than that of 1878. The trans-mountain movement of these storms indicates that a storm similar to that of July 14–16, 1916, could cross the Appalachians and produce a rainfall diminished but slightly from that which actually occurred in North Carolina, namely, 13.0 in. over 5,000 sq miles.

S. M. BAILEY *and* G. R. SCHNEIDER,
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Louisville, Ky.

The Correct Care of Surveying Instruments

DEAR SIR: The article on "The Care of Surveying Instruments" by L. H. Berger, in the July issue, contains considerable information that should be of value to all engineers using surveying instruments, especially those who must of necessity use instruments in remote and inaccessible locations. The writer's experience with the telescopic alidade used in most plane-table work has been rather extensive and thoroughly bears out Mr. Berger's statement that "frequent cleanings seem to be the best preventive of instrument trouble." Inexperienced men frequently hesitate to undertake this cleaning and its attendant oiling and as a result, especially when the instrument is used in a dusty location, the trunnions and their bearing surfaces on the standards become coated with a dust and oil cake that completely destroys the quality of the bearing and makes use of the instrument very unsatisfactory. Usually in such a case the instrument is blamed for troubles that are the fault of the instrumentman and that could easily be prevented or corrected if the instrument were cleaned and oiled.

It is doubtful if instrument-makers realize the amount of rough use that the average telescopic alidade receives. The alidade is usually carried on the outside of the plane-table case on the topographer's back. This does not subject the alidade to any particular rough use unless the topographer falls or unless he is going through heavy brush or undergrowth. In the latter case it is practically impossible to prevent the alidade from receiving occasional blows from small twigs or, sometimes, from limbs or considerable size. A few days or even a few hours spent in following a topographer through a heavy brush field would be very instructive to an instrument designer and might lead to a change in design. The desirable changes appear to be in a strengthening of the standards and of the springs which limit the vertical motion of the alidade. On one well-known make of alidade, which the writer used, these springs were almost continually so bent as to be useless, though every effort was made to treat the instrument with the care which it undoubtedly deserved.

The Johnson head tripod is an important part of the plane-table outfit and is frequently the source of considerable trouble to the inexperienced man. As in the case of other instruments the best preventive is frequent and regular cleaning, though oil should never be used. The surfaces of the tripod, both interior and exterior, become coated with dust, causing the tripod to have a jumpy irregular motion and making satisfactory orientation and leveling of the board very difficult. If the head is not cleaned when this condition develops the tripod may freeze and become seriously damaged. However, this can easily be prevented by cleaning the tripod surfaces. Mere wiping of these surfaces with a cloth does not seem to be sufficient, and rubbing with fine steel wool or, in an emergency, with fine sand paper is necessary to bring the tripod head to proper condition. The writer would be glad to know of some less drastic method which will accomplish the same result.

As an example of what an inexperienced man may do the writer recalls one case in which a topographer broke a level vial and, not knowing that he had a replacement in his instrument box, made a trip of several hundred miles by automobile to obtain a new vial. Whenever an inexperienced man is required to use and care for an instrument he should be taught the principles of instrument care.

ROBERT H. GRIFFIN, Jun. Am. Soc. C.E.
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Strengthening the Williamsburg Bridge

DEAR SIR: I have read with much interest Mr. Hovey's article entitled, "A Bridge Builder Looks Back," in the August issue of CIVIL ENGINEERING. He mentions the fact that one of the two most difficult engineering problems he ever had was in connection with the strengthening of the Williamsburg suspension bridge in 1914, without stopping traffic.

Having been connected with the strengthening of the end spans of this bridge, I was assigned by the late Austin Lord Bowman, M. Am. Soc. C.E., chief engineer of the Department of Bridges, on the extensometer investigations made in cooperation with the New York City Department of Bridges and the National Bureau of Standards under the auspices of James E. Howard, engineer physicist.

The article was of special interest to me as it distinctly recalled not only the difficult problems involved but also the value of the extensometer measurements taken on this important work.

The fact of the matter is that this was the first time in the history of the Department of Bridges that extensometer investigations were ever conducted on one of its bridges, and particularly on a structure of such magnitude.

In passing it occurs to me that it would be of material interest for readers to refer to the discussion by Isidore Delson, Assoc. M. Am. Soc. C.E., on the paper entitled, "Stress Measurements on the Hell Gate Arch Bridge" (TRANSACTIONS, Am. Soc. C.E., Vol. 28, pages 1087–1089). In this Mr. Delson discusses the value of the extensometer measurements—particularly in one of the wedging operations—which solved a critical and perplexing erection problem, not only in the close agreement of the computed stresses, but also in the fact that after the removal of the pins there was no apparent change in the stresses of the members.

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Experiments on the Hydraulic Jump

TO THE EDITOR: Fred C. Scobey's article, "Notes on the Hydraulic Jump," in the August issue, is a most welcome addition to the literature of the subject, representing as it does a valuable summary of the practical conclusions derived by the author from his many years of work in this field. The writer ventures to add a few words that may help to clarify some of the points on which Mr. Scobey expresses the need for further demonstration and proof.

There is abundant evidence that Mr. Scobey is correct in stating that no backwater effect travels upstream in shooting flow. Small standing waves form for a short distance upstream from obstacles introduced into a steam flowing with higher than critical velocity, but these small waves are not "backwater effects" in the usual sense of the word, and the water-surface profile shows no far-reaching upstream effect as it would in the case of streaming flow. Quantitative evidence is provided by tests of broad-crested weirs under partial submergence. If the submergence is less than the critical depth, the tailwater level remaining below the upper limit of the shooting flow, the discharge is unaffected by the submergence. Thus Mr. Scobey's statement, "if critical flow holds at a crest, then tailwater stage, below the jump, has no effect on the discharge over the crest," is seen to be true, even when the jump is drowned out by tailwater higher than the crest by an amount equal to the critical depth. This will not be so, however, if the top of the weir is not sufficiently broad to stabilize the position of the jet.

The statement that "shooting flow cannot well be gradually slowed down" must be questioned. Both S. M. Woodward (Miami Conservancy District Technical Reports, Part III) and B. A. Bakhmeteff (*Hydraulics of Open Channels*) give several cases of backwater curves in which shooting flow is gradually slowed down.

Besides the experiments on the jump in sloping channels listed by Mr. Scobey, there is a very extensive large-scale set of experiments, which were just finished by the late D. L. Yarnell at the time of his death. In the writer's opinion, Mr. Yarnell's tests were complete enough to afford a satisfactory answer to the problem. It is to be hoped that the U. S. Department of Agriculture will soon make the results of these tests available to the engineering profession.

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CIVIL ENGINEERS' SALARIES

Grading Plan and Compensation Schedule for Civil Engineering Positions

Lower Professional Grades, Including Pre-Professional

As Unanimously Adopted by the Board of Direction, American Society of Civil Engineers, July 25, 1939

FOREWORD

Professional civil engineering positions may be grouped according to relative importance of duties to be performed and of the responsibilities accorded thereto. The three lowest grades for professional service are herewith presented and they are numbered in order from lowest to highest. For each grade there is a general specification intended to define as well as describe the duties and responsibilities usually associated with the grade and the expected adaptability of the person to properly perform the work and assume the responsibilities so specified for that grade.

While the specifications indicate a range for duty and responsibility in each grade, to be of general application to civil engineering employment, these specifications should be used with liberal judgment as to application and adaptability to meet local situations and sectional divergence which naturally exist within a field national in extent. *Recognition must therefore be taken of possible variance in the social and economic status of engineering employment and compensation rates as they may prevail when direct comparison in their application is made of prevailing conditions in one local area with those of another.* When so used, the specifications, examples and compensation rates will adequately serve as a guide to establish the basic organization requirement as well as the applicable remuneration for the several grades.

Pre-professional civil engineering positions may be grouped according to relative importance of duties and responsibilities. The group or range is herewith presented by specification, example and compensation rate. Its use and adaptability should be based on the same prerogatives as accorded to the lower professional grades.

Pre-Professional Grade 0

A. *Definition:* To include all positions the duties of which involve the performance of work incident, subordinate, or preparatory to work performed by employees holding positions in the professional grades.

For purpose of application in the field of civil engineering practice pre-professional positions are classified under one grade, but recognition given the appointee based on education, experience and preliminary technical training.

B. *Compensation:*

1. Starting rate—$110.00 a month, $1320.00 a year.

2. Full rate—$165.00 a month, $1980.00 a year.

Note: (1) The scale or range for Grade 0 has ten intermediate rates, or twelve rates for the scale, separated by intervals or step-ups of $60.00 a year, or $5.00 a month.

(2) In practice it is contemplated that the new employee who has completed the twelfth (high) school grade will be paid the starting rate; and that the new employee who has graduated with a degree in civil engineering from an accredited college or university, will be paid an intermediate rate. The first increase should be made within a reasonable time after entering the service upon demonstrated ability for the work and acceptability to the organization.

C. *Requirements:* The requirements for education, experience and skill for positions in pre-professional engineering service will be of any degree inferior to those defined under Grade I. The duties to be performed under this classification shall range from the simplest routine work to that of a knowledge within a limited field of civil engineering work, requiring the exercise of reasonable judgment appertaining to its completion.

IN February 1939, "Civil Engineering" published, for appraisal and discussion, a Grading Plan and Compensation Schedule for Civil Engineering Positions, prepared by the Society's Committee on Salaries, which had been presented to the Board of Direction at its January 1939 meeting. In succeeding weeks 24 Local Sections, with memberships totaling approximately 7400, submitted reports, and in addition, written comments were received from numerous individual corporate members.

At the April 1939 meeting of the Board, a special committee of three Directors was appointed to correlate the comments, suggestions, and criticisms received, and to report back to the Board with recommendations.

Pursuant to these instructions the Special Committee presented, at the July meeting of the Board, a redrafted Grading Plan and Schedule which it recommended for adoption or approval. The Committee added to the earlier report a section relating to pre-professional work. This Plan and Schedule were unanimously adopted July 25, 1939.

Professional Grade I

A. *Definition:* To include all positions the duties of which involve civil engineering work of minor professional difficulty and responsibility.

B. *Compensation:*

1. Starting rate—$170.00 a month, or $2040.00 a year.

2. Full rate—$210.00 a month, or $2520.00 a year.

Note: (1) The scale or range for Grade I has three intermediate rates, or five rates for the scale, separated by intervals or step-ups of $120.00 a year, or $10.00 a month.

(2) In practice it is contemplated a new appointee with minimum qualifications for the position will be paid the starting rate established for the grade.

C. *Requirements:* The positions are described with respect to minimum qualifications requisite for satisfactory performance of characteristic duties, as follows:

1. *Education and experience:* (a) Graduation with a degree in civil engineering from an engineering

school on the accredited list of the Engineers' Council for Professional Development and in general the equivalent of at least two years' experience on engineering work, or (*b*) completion of the twelfth (high) school grade, *additional self-improvement through study of technical engineering subjects*, and in general the equivalent of at least eight years' experience on engineering work, or (*c*) any equivalent combination of education and experience.

2. *Knowledge and skill:* Good working knowledge of mathematics; of the properties of materials; of the customs and practices followed in expressing engineering decisions in drawings; of the elementary work required for investigation and development of projects and their design, construction, inspection, testing, operation and maintenance; of the elements of engineering research; of the use, care and adjustment of applicable engineering instruments and tools; and skill and dexterity in subject matters and techniques involved in such work commensurate with general knowledge and skill requisite for this grade.

3. *Applicable or characteristic duties:* Under full technical direction to follow specific, detailed or frequent instruction applicable to any field of civil engineering practice and type of organization in the performance of work allocated to this grade, of which the following examples are typical.

(*a*) *In surveying:* Individually, or with one or more assistants, making observations and gathering and recording data for engineering reports, the preparation of plans and other technical uses; doing field work involving the use of transits, levels and other surveying instruments and equipment including their proper operation and adjustment; assisting in computing, compiling and platting of field data; and having charge of a sub-party and being responsible for the results of the work performed by the party.

(*b*) *In designing and drafting:* Assisting in laying out and preparing detailed plans and estimates for engineering projects; making compilations of engineering statistics; preparing maps, graphs, charts, diagrams and tables for purpose of interpretation and representation of data for engineering purposes and uses.

(*c*) *In construction:* Ability to interpret plans and specifications and to stake out the work; make field tests of materials; select samples of materials for laboratory tests; inspection of contract work; rejection of non-specification work and materials; and the preparation of reports and estimates.

(*d*) *In general:* Doing work of equal difficulty and responsibility involving in part two or more of the aforementioned civil engineering duties, with or without other engineering work of like grade, or work of pre-professional, or of clerical or other non-engineering character.

4. *Duties by fields of practice:* As a member of an engineering organization qualified to do such work as indicated in several of the more usual specialized fields of civil engineering practice as may be illustrated by the following:

(*a*) *In highway engineering:*

(1) On preliminary location, or final road surveys, establishing lines, taking cross-sections, making property and drainage surveys, taking soundings for structures, establishing lines, grades, angles, and contours; and establishing or verifying records of alignment, boundaries of rights of way and ownership and limits of abutting properties.

(2) Making plans involving center lines, grades and minor details; designing typical cross-sections; preparing preliminary and final quantity estimates; and computing costs.

(3) Supervising and inspecting the construction of roads, bridges, and drainage structures. Assisting generally on office and field work involved in securing rights of way for highway projects, as by mapping proposed locations and boundaries from survey field notes; making title searches; verifying legal descriptions; preparing right-of-way maps and descriptions of parcels of land required.

(4) Doing work of equal difficulty and responsibility involving two or more of such classes of duties, or other civil engineering processes, with or without other engineering work of like grade or of pre-professional, or of clerical or other non-engineering, character.

(*b*) *In structural engineering:*

(1) Developing or checking plans and estimates for structures such as culverts, floor slabs, small plate girders, and the like, or preparing detailed plans according to prescribed designs for certain parts of elements of structures such as members of truss bridges, columns in steel buildings, floor beams, and the like.

(2) Supervising and inspecting the construction of structural works and assisting generally on office and field work, such as interpreting the plans, staking out the work, field tests of materials, and the preparation of reports.

(3) Doing work of equal difficulty and responsibility involving two or more of such classes of duties, or other civil engineering processes, with or without other engineering work of like grade or of pre-professional, or of clerical or other non-engineering, character.

(*c*) *In hydraulic engineering:*

(1) Detailing elements or features of projects the main features of which have previously been determined from sketches and computations, for example, establishing the grade of a trunk-line sewer of which the location, size, and shape have been determined, designing intersections with trunk lines, and determining the grades of lateral sewers.

(2) Having charge of location or other surveys for canals, waterways, dams, locks, pipe lines, or any of them. Supervising and inspecting the construction of hydraulic works and assisting generally on office and field work.

(3) Doing work of equal difficulty and responsibility involving two or more such classes of duties, with or without other engineering work of like grade or work of pre-professional, or of clerical or other non-engineering, character.

(*d*) *In sanitary engineering:*

(1) Designing and preparing plans and estimates for parts of sewerage systems or elements of sewage treatment plant work of which the basic designs, sketches and calculations are available.

(2) Supervising and inspecting the construction of sanitary works and assisting generally on office and field work, such as interpreting the plans, staking out the work, field tests of materials, and the preparation of reports.

(3) Doing work of equal difficulty and responsibility involving two or more of such classes of duties or other civil engineering processes, with or without other engineering work of like grade, or work of pre-professional, or of clerical or other non-engineering, character.

(*e*) *In research and testing:* The making of routine tests of materials and equipment, the reading of instruments, recording, computing and compiling of test data, and the collecting, computing and compiling of laboratory and research statistics.

(*f*) *In general:* Doing work of equal difficulty and responsibility involving two or more of the fields of practice described, or in other civil engineering fields of practice, with or without other engineering work of like grade, or work of pre-professional, or clerical or other non-engineering, character.

Professional Grade II

A. *Definition:* To include all positions the duties of which involve civil engineering work of intermediate professional difficulty and responsibility.

B. *Compensation:*

1. Starting rate—$220.00 a month, or $2640.00 a year.

2. Full rate—$270.00 a month, or $3240.00 a year.

Note: The scale or range for Grade II has four intermediate rates, or six rates for the scale, separated by intervals or step-ups of $120.00 a year, or $10.00 a month. Also see Note (2) Grade I.

C. *Requirements:* The positions are described with respect to minimum qualifications requisite for satisfactory performance of characteristic duties, as follows:

1. *Education and experience:* The same as for Grade I and additional thereto the equivalent of at least two years' experience on applicable engineering work.

2. *Knowledge and skill:* Familiarity with mathematics, with physics, with the properties of materials used in construction, with engineering specifications, with the customs and practices followed in expressing engineering decisions in drawings, and with the important sources of engineering information; more than ordinary skill in the application of such knowledge to engineering problems; ability to organize engineering data and to draw logical conclusions therefrom and to present facts, findings, and definite conclusions clearly and concisely; under supervision, ability to program, plan, organize, supervise, and check the work of others, and to lay out the working details of projects; and skill and dexterity in subject matters and techniques involved in such work commensurate with general engineering knowledge and skill requisite for this grade.

3. *Applicable or characteristic duties.* Under general technical direction to perform and be responsible for the technical soundness of methods and the accuracy of results for work applicable to any field of civil engineering practice and type of organization such as those described for Grade I—(3-*a*, *b*, *c*).

4. *Duties by fields of practice:* As a member of an engineering organization qualified to design, check and approve plans, supervise construction, to prepare estimates and reports, and to serve as leader of a group in any field of civil engineering practice and type of organization such as those described for Grade I—(4-*a* to *f*).

Professional Grade III

A. *Definition:* To include all positions the duties of which involve civil engineering work of considerable professional difficulty and responsibility.

B. *Compensation:*

1. Starting rate—$280.00 a month, or $3360.00 a year.

2. Full rate—$325.00 a month, or $3900.00 a year.

Note: The scale or range for Grade III has two intermediate rates, or four rates for the scale, separated by intervals or step-ups of $180.00 a year, or $15.00 a month. Also see Note (2) Grade I.

C. *Requirements:* The positions are described with respect to minimum qualifications requisite for satisfactory performance of characteristic duties, as follows:

1. *Education and experience:* The same as for Grade II and additional thereto the equivalent of at least three years' experience on applicable engineering work.

2. *Knowledge and skill:* Thorough familiarity with mathematics, with physics, with the properties of materials used in construction, with engineering specifications, with the customs and practices followed in expressing engineering decisions in drawings, and with the important sources of engineering information; a high degree of skill in the application of such knowledge to engineering problems; ability to organize engineering data and to draw logical conclusions therefrom and to present facts, findings, and conclusions clearly and concisely; ability to program, plan, organize, supervise, and check, the work of others, and to lay out the working details of projects of considerable scope; and skill and dexterity in subject matters and techniques involved in such work commensurate with general engineering knowledge and skill requisite for this grade.

3. *Applicable or characteristic duties:* Under occasional technical direction to perform and be responsible for the technical soundness of methods and the accuracy and reliability of the results for work applicable to any field of civil engineering practice and type of organization such as those described under the corresponding heading for Grade I and Grade II.

4. *Duties by fields of practice:* As a member of an engineering organization fully qualified and responsible for the planning and the supervision of projects applicable to any field of civil engineering practice and type of organization such as those described under the corresponding heading for Grade I and Grade II.

SOCIETY AFFAIRS

Official and Semi-Official

British American Engineering Congress Canceled

International Situation Forces Last Minute Abandonment of Plans; Society Will Hold No Fall Meeting

CANCELLATION of all plans for the British American Engineering Congress, scheduled to open in New York on September 4, was determined upon late Friday, August 25, following receipt of cablegrams from London, from the Institution of Civil Engineers and the Institution of Mechanical Engineers, announcing that they both were forced to withdraw from participation on account of the international situation.

The decision was reached with deep regret at a conference between officials of the American Society of Civil Engineers and the American Society of Mechanical Engineers after a phone consultation with the Engineering Institute of Canada, the fifth participating group. As a result, there will be no Fall Meeting of the Society of any sort. There will be the usual fall meeting of the Board of Direction, which must be held in order to transact certain items of business that cannot be postponed until the Annual Meeting.

Postcards announcing this development were mailed on August 26 to the American and Canadian membership of the Society.

Constitutional Amendments Re Dues Considered at San Francisco Meeting

IN accordance with Constitutional requirements, a meeting of the Society was held on Wednesday morning, July 26, 1939, at the time of the Annual Convention in San Francisco, Calif., being called to order immediately following the general session. President Sawyer presided over the meeting.

Under the head of unfinished business, the proposed amendment to Article IV of the Constitution was considered, this having to do with the reduction of dues of members resident in District No. 1 and with the compounding of dues. On motion from the floor, duly seconded and carried, the President was instructed to appoint a committee of five Corporate Members, one resident in each of the four zones and one, representative of the Society at large, to be the chairman. None are to be members of the present Board of Direction.

On further motion, duly seconded and carried, the proposed Constitutional amendment was referred to this Committee, the Committee to report at the Annual Meeting to be held in January 1940 in New York, on the following four specific points:

1. Whether there is justified any differential in dues between the members in District No. 1 and elsewhere;
2. If so, what differential should be provided;
3. If not, is any general or specific increase in dues required to offset the loss in revenue of the Society; and
4. The schedule of dues which would be in the best interest of the Society.

There being no other business the meeting was adjourned.

Mead Prizes for Younger Engineers

BY ACTION of the Board of Direction at its San Francisco meeting, the necessary approval was given the establishment of prizes for papers having to do with the ethics of engineers. These prizes are to be awarded through the generosity of Past-President Daniel W. Mead and are available only to younger engineers.

Long an exponent of higher standards for engineers in their dealings among themselves and with the public, Dr. Mead through his studies, his talks, and his teachings has become known as an authority on ethics. His textbooks also have included such discussions and are among the few such treatments now available to engineers. It is natural, therefore, that he should take the lead in bringing this matter still more forcefully before the profession.

His ideas, now officially endorsed and given the necessary authorization by the Board, contemplate two yearly prizes of $50 and $25. Competition is to be among younger engineers and is confined to Juniors and members of Society Student Chapters. Awards are expected to be made on the basis of papers, dealing with ethics, presented before Local Sections, Division meetings, or possibly Student Chapter or Local Section conferences. It is further proposed that conduct of the competition and determination of the awards shall be under the Society's Committee on Professional Conduct.

As soon as the official rules are drawn up and approved, a more complete announcement will be made. Although not eligible for the prizes, older engineers will take an active interest in this competition.

Two Honorary Members Elected

IN accordance with constitutional procedure, two distinguished foreign engineers were elected to honorary membership in the Society at the San Francisco meeting of the Board of Direction, July 24, 1939. One is William James Eames Binnie, of London, England; the other, John Morrice Roger Fairbairn, of Montreal, Que. Mr. Binnie has been a member of the Society since 1929, and Mr. Fairbairn since 1913.

Since the death of his father in 1917, Mr. Binnie has been senior partner of the firm of Sir Alex. Binnie Son and Deacon, whose practice is chiefly in connection with sanitary works. He is now serving as president of the Institution of Civil Engineers, and has previously held the same position in the Institution of Water Engineers and the Institution of Sanitary Engineers.

A lifetime of service with the Canadian Pacific Railway is the record of Mr. Fairbairn, who retired this year from his post as chief engineer. He is a past-president of the Engineering Institute of Canada, and of the American Railway Engineering Association.

Return of "Factual Survey" Cards Urged

MEMBERS, Associate Members, and Juniors who have not yet returned the "Factual Survey" cards sent them in July are urged to do so at once. To date about half have responded, but it is hoped that a much larger percentage of the membership can be included in the final tabulations and analyses.

The survey, as announced in the July issue of CIVIL ENGINEERING, is designed to supply certain basic facts essential to the work of the Society in improving the social and economic status of its members. It is being sponsored by the Committee on Professional Objectives and the Committee on Salaries. The cards request data on work, age, grade of membership, position, and income, and are not to be signed.

San Francisco Entertains Record-Breaking Convention

Outstanding Technical Sessions Vie with Fair in Making Meetings of July 26–28 a Great Success

MAKING glowing predictions for any future event is somewhat hazardous. Yet rarely have such anticipations been exceeded so generously as at the San Francisco Convention. The time was thought to be propitious; the program seemed excellent; the Fair was expected to be an attraction; and the local Society organization was known to be efficient. So they proved, all of them—and much more.

It is easy to place false emphasis on size. Mere numbers are hardly a measure of quality. On the other hand a generous attendance indicates interest; it ensures a response to speakers; it makes the informal chats and group conversations in the corridors a multiplied pleasure; and the same is true for the planned social events—not to mention the additional advantage that the problem of financing is immeasurably simplified.

Both quantitatively and qualitatively the San Francisco Convention must be accounted a glowing success. The over-all registration well exceeded 1,600, thus setting a new mark for Society meetings outside of New York. The individual sessions were on a similar scale. This created problems of arrangement. In one instance it became necessary to interchange meeting rooms during the technical sessions. The lesser group willingly assented to being crowded into a smaller hall in order that the larger session might have adequate room.

The Technical Division sessions were held on Wednesday afternoon, on Thursday morning, and on Friday morning and afternoon. On Wednesday afternoon four Divisions met. The Hydraulics and Irrigation Divisions held a joint session for the hearing of three papers—on the hydraulics of sprinkling systems for irrigation, silt problems of the Imperial Irrigation District, and the design of curves for high-velocity open channels. The Waterways Division session dealt with the water supply of Chicago, the controlling works for the Chicago River, and the Central Valley project in California. The Soil Mechanics and Foundations Division held two sessions, the first on Wednesday afternoon and the second on Thursday morning. The papers on Fort Peck, originally scheduled for Thursday were transferred to Wednesday, exchanging with those on the photoelastic analysis of stress in earth masses, the effect of earthquake motion on foundations, and the progress report of the Committee on Earth Dams and Embankments. This was a fortunate rearrangement, as it gave ample time for active and extended discussions immediately following the Fort Peck presentations. The paper on sealing the clay lining of the lagoon at Treasure Island was given on Thursday as originally scheduled.

The Irrigation and Structural Divisions also met on Thursday morning. The program of the former included three main papers—on concrete linings for irrigation canals, the water supply for the Central Valley project, and the permissible quality of irrigation water. In the latter the general subject was earthquake resistant design, on which four papers were presented.

As Thursday afternoon was devoted to luncheon and sightseeing at the Fair, the next technical sessions came on Friday, when Sessions III and IV of the Power Division's extensive symposium on Masonry Dams were presented throughout the day. Also on Friday, the Hydraulics Division and the Sanitary Engineering Division held morning sessions. The former had on its schedule a paper dealing with the uses of flow and pressure nets in hydraulic investigations and one on flood experiences of the Southern Pacific Railroad. The Sanitary Engineering Division considered aspects of watershed and reservoir sanitation; the Minneapolis-St. Paul chemical treatment plant; the sanitary fill method of refuse disposal at San Francisco; the water supply system of San Francisco; and the Richmond-Sunset sewage treatment plant.

On Friday afternoon the City Planning and Highway Divisions, in joint session, heard three papers—on roadside control through zoning, metropolitan highway problems, and financial and physical plans. At this time also the Sanitary Engineering Division made an interesting excursion to points of special interest, concluding with a supper at the picnic grounds near Redwood City. During the day the Structural Division also conducted a popular inspection trip, including the monumental bridges and other features.

The Golden Gate International Exposition more than fulfilled all expectations. Perhaps some out-of-town members were inclined to forego the advantages of technical meetings to visit nearby Treasure Island. If so, their absence did not noticeably affect attendance. One reason for this was undoubtedly the excellent plans to include the Fair in the official program. Shortly after noon on Thursday, immediately following the morning sessions, the exodus from headquarters began. By special bus, by ferry, and by private car they left for the Fair and in such numbers that the dining facilities provided there both for lunch and for dinner were taxed far beyond expectations. The schedule left the afternoon and part of the evening for sightseeing.

By night, the Fair took on an aspect even more beautiful than by day. The fountains, courts, and buildings stood out in colorful splendor, dominated by the graceful Tower of the Sun as a central point of focus. Many an engineer explored the art exhibits, the various national and state buildings, and the showing of natural as well as industrial resources. Then too, if the truth be told, not a few took advantage of the opportunity to investigate the attractions of the Gay Way.

Because of the manner in which the program was arranged, social events at the St. Francis Hotel, headquarters for the meeting, were relatively few. The most notable was the dinner and dance on Wednesday evening, largely attended and thoroughly enjoyed. Following the splendid repast, the program was limited to a number of introductions and a few short informal speeches—an arrangement designed to permit the enjoyment of a correspondingly longer period of dancing thereafter.

A number of valuable supplementary meetings, held during the week, deserve a word. The Local Sections Conference met all day Tuesday for a helpful exchange of views; a more extended report of this meeting is expected to be available for a later issue. Various Board of Direction and Division committees met either before or during the Convention proper. The official summer meeting of the Board, with large attendance, extended through morning and afternoon sessions Monday and Tuesday. At the other end of the week, Friday and Saturday, the National Council of State Boards of Engineering Examiners held its sessions. For the six days preceding the convention week an enthusiastic party traveling by special Pullman and auto buses was enjoying a pre-arranged tour. Lake Tahoe, Mono Lake, Yosemite Park, and the Mariposa Grove were significant points in the itinerary.

That the civil engineer is held in high esteem in San Francisco and the West can be quickly sensed by a visitor. Easily the dominating permanent features of the whole region are the two immense bridges giving easy access from a great city to areas east and north. By such means the Fair is made accessible to the Bay regions on both sides. Thus, too, these structures have affected the transportation, the economics, and the everyday thinking of all citizens. Visiting engineers, as they continuously saw or traveled over these monumental structures, experienced a renewed pride in their profession.

Local residents made no comments on the weather, but to most of the visitors, especially those who had experienced the full heat of summer in central California or in the East, it seemed too good to be true. An overcast sky in the morning usually gave way to a sunny noon, which in turn was sometimes followed by billowing fog from the Pacific in the late afternoon. No one complained of the heat, but one or two claimed that they found it too cold. The planning of meeting trips and excursions is greatly simplified by such a climate. Other prospective Convention cities please note!

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To the ladies it seemed that the week was one continuous round of entertainment. It began, as regards the advance registrants, as early as Sunday afternoon, and it continued with few intermissions until Friday afternoon. Trips to the Fair were interspersed with teas, rides, and sightseeing tours. One event, on Friday, following a fine lunch in Palo Alto and a visit to Stanford University, was the more delightful because it came unexpectedly. Mrs. Herbert Hoover entertained at tea in her lovely home on the University campus. This was not only the last, but to many the crowning event on the ladies' program.

Local officers and committee members had all meeting events under continuous and perfect control. They were gracious in saying that they too enjoyed the meeting. The enjoyment that comes from satisfaction in a job well done they certainly must have had. No successful meeting "just grows"; it results from long and careful preparation and hard work on the part of many individuals. The local committees, men and women, have the thanks of every one in attendance for a fine week and a splendid convention, without a single hitch.

Hilgard Prize to Recognize Hydraulic Papers

IT has been previously noted in the August issue that the Society has been made the beneficiary of a bequest from the late Karl E. Hilgard, M. Am. Soc. C.E., of Zurich, Switzerland. The purpose of this provision by Dr. Hilgard was to establish a prize in hydraulics, to be administered by the Society.

When this matter was presented to the Board of Direction at its meeting in San Francisco, the Board accepted the responsibility with appreciation of its significance and value. At the same time the Board approved the specific provisions whereby the prize is to be awarded. Notation in the Society Year Book, including the rules now made effective by Board action, will appear as follows:

"*Hilgard Prize.* This prize was instituted in 1939 as a result of an endowment left to the Society for the purpose by the late Karl E. Hilgard, M. Am. Soc. C.E., of Zurich, Switzerland.

"I. The prize shall consist of a cash award of $50.

"II. The prize may be awarded not more often than once in two years.

"III. Papers in any of the volumes of TRANSACTIONS for the biennium under consideration are eligible. The award shall be given to the author or authors of that paper which is judged to be of superior merit, dealing with a problem of flowing water, either in theory or practice. Preferably the award shall be given to a paper which is not otherwise recognized by receiving a Society prize. The value of the paper shall be judged both on the basis of the subject matter and the method of presentation.

"IV. The Committee on Prizes each year shall consider the possibilities for the Hilgard Prize, whether or not it is to be awarded that year; and shall make recommendation accordingly.

"V. The Rules of this award and the cash value of the prize are subject to the administration of the Board of Direction."

Appointments of Society Representatives

ROBERT B. BROOKS, M. Am. Soc. C.E., has been appointed the Society's representative to the Engineering Congress to be held at Wellington, N.Z., in February 1940, under the auspices of the New Zealand Institution of Engineers as part of the celebration in connection with the centenary of the official founding of New Zealand.

JAMES L. FEREBEE, M. Am. Soc. C.E., was appointed to represent the Society at the annual meeting of the National Council of State Boards of Engineering Examiners, which was held in San Francisco, Calif., July 26–29, 1939.

DONALD H. SAWYER, President Am. Soc. C.E., has been appointed one of the Society's representatives on the John Fritz Medal Board of Award for the four-year term, October 1939 to October 1943.

JOSEPH B. TIFFANY, JR., Jun. Am. Soc. C.E., has been appointed a member of the Society's Committee on Hydraulic Research for the duration of his connection with the U. S. Waterways Experiment Station at Vicksburg, in place of Capt. Paul W. Thompson, formerly director of the Station, who has been transferred to another assignment.

Tellers' Report on First Ballot for Official Nominees

To the Secretary
American Society of Civil Engineers:

New York, N.Y.
August 1, 1939

The tellers appointed to canvass the First Ballot for Official Nominees report as follows:

For Vice-President, Zone I		
George Latimore Lucas	441	
Charles Milton Spofford	327	
Scattering	34	
Blank	6	
Total		808
For Vice-President, Zone IV		
Joseph Jacobs	347	
John Cyprian Stevens	233	
Scattering	95	
Blank	21	
Total		696
For Directors, District 1 (two to be elected)		
Ernest Payson Goodrich	254	
Lazarus White	248	
Ineligible Candidates	101	
Scattering	169	
Total votes	772	
Blank	264	
Total		1,036
Actual number of ballots received (1/2 of above figure)		518
For Director, District 2		
Clarence Moore Blair	48	
Charles Milton Spofford	6	
Ineligible Candidate	20	
Scattering	34	
Blank	55	
Total		163
For Director, District 6		
Clifford George Dunnells	87	
Scattering	15	
Total		102
For Director, District 10		
Armour Cantrell Polk	175	
John Franklin Reynolds	94	
Scattering	15	
Blank	1	
Total		285
For Director, District 13		
Charles Gilman Hyde	98	
Leon Hagop Nishkian	14	
Ineligible Candidate	15	
Scattering	40	
Blank	24	
Total		191
Ballots canvassed		1,891
Ballots withheld from canvass:		
From members in arrears of dues	102	
Without signature	7	109
Total number of ballots received		2,000

Respectfully submitted,

RALPH H. MANN, *Chairman*

A. C. Seaman	Bernard L. Weiner
Wm. J. Delaney	Howard Holbrook
Gilbert C. Whitney	Edward N. Whitney
Sigmund Roos	Charles D. Thomas
Bernard Gaber	G. Birger Andreen
Clifton W. Bolieau	F. V. Hayes
Louis H. Lockwood	Alexander A. Dedouloff

Tellers

Meeting of the Board of Direction—Secretary's Abstract, July 24–25, 1939

ON JULY 24 and 25, 1939, the Board of Direction met at the St. Francis Hotel, San Francisco, Calif., with President D. H. Sawyer in the chair, and Secretary Seabury and the following members of the Board in attendance: Past-Presidents Mead and Riggs, Vice-Presidents Noyes, Pirnie, and Ferebee; and Directors Agg, Anderson, Ayres, Bres, Brooks, Davis, Dean, DeBerard, Harrington, Hudson, Leeds, Legaré, Lewis, Needles, Root, Sawin, Shea, and Stanton.

In Memoriam

The Board stood for a moment of silence in respect to its late fellow member, Director Tiffany, who died on June 1, 1939. A committee was appointed to prepare suitable resolutions. Subsequently the Board approved the committee's suggestion in the form as given in the August issue, page 513.

Approval of Minutes

Minutes of the Executive Committee of the Society for April 16, 1939, and of the Board for April 17–18 were discussed and approved; the actions of the Executive Committee were adopted as actions of the Board.

New Director

With regard to the vacancy in the Board caused by the death of Director Tiffany, it was reported that the four Sections in the affected District, No. 12, had unanimously endorsed and recommended Glenn L. Parker of Tacoma, Wash., for this vacancy. Mr. Parker was unanimously elected to fill the unexpired term, was presented to the Board, and took his place in all succeeding deliberations.

Local Sections Revise By-Laws

After adoption by the Sections and reference to the Board, the By-Laws of the Oregon Section and the Constitution and By-Laws of the Syracuse Section were approved as revised.

J. Waldo Smith Fellowship on Hydraulics

The committee on the J. Waldo Smith Fellowship on Hydraulics recommended appointment for 1939–1940 of Walter J. Meditz of the Polytechnic Institute of Brooklyn. The Board confirmed the nomination and Mr. Meditz was appointed, as noted more fully in the August issue, page 513.

Hilgard Hydraulics Prize

The Board adopted a resolution establishing "The Hilgard Prize," financed by a bequest from the estate of the late Karl Emil Hilgard, M. Am. Soc. C.E., of Zurich, Switzerland. Rules for the prize, as adopted, will be found elsewhere in this issue.

Financial Matters

Various matters affecting the current financial budget were considered, with discussion and appropriate action.

New Honorary Members

Ballots were canvassed for Honorary Members, resulting in the election of J. M. R. Fairbairn of Montreal and W. J. E. Binnie of London. Brief accounts of these eminent engineers appear in a separate item elsewhere in this issue.

Professional Objectives

The Board adopted six recommendations of the Committee on Professional Objectives relating to maintainance of Society membership, as reported in an adjoining item.

Membership

Considerable study was given to applications for membership, as submitted through the Committee on Membership Qualifications. After thorough discussion, action was taken on all the cases presented.

Communications

The Board authorized a letter of greeting to be addressed to Charles D. Marx, Hon. M. Am. Soc. C.E., of Palo Alto, Calif., the oldest living Past-President.

It also wired its regrets to another afflicted member, Ralph J. Reed, of Los Angeles, a former Director. (Mr. Reed died two days later, July 27.)

Seismological Studies

Upon receipt of a report from the Technical Procedure Committee, the Board endorsed in principle efforts to further seismological knowledge and asked the Committee to keep it informed as to further developments.

Mead Prizes

On recommendation of the Committee on Professional Conduct, the Board approved the establishment of two prizes dealing with ethics, sponsored by Past-President Daniel W. Mead. A separate item on another page gives more complete information.

Compensation for Civil Engineers

Following the report of the Committee on Salaries as published in the February CIVIL ENGINEERING, the Board appointed a committee to correlate discussion of this matter. As a result the Committee presented to the Board for action a revision of the former report. After full discussion and minor changes, the report, with its "Grading Plan and Compensation Schedule for Civil Engineering Positions" was adopted. Copy of this report in full will be found on pages 566–568.

Other Matters

Reports from standing committees and Society representatives were received and discussed, with appropriate action; and similarly with other administrative matters.

Adjournment

The Board adjourned to meet in New York, N.Y., on Saturday, September 2, 1939.

Forecast for September "Proceedings"

COMBINING GEODETIC SURVEY METHODS WITH CADASTRAL SURVEYS

By Carl M. Berry, Assoc. M. Am. Soc. C.E.

Advocates a complete reversal of the classical "ground methods" of coordinating points in a geodetic survey.

DEVELOPMENT OF THE COLORADO RIVER IN THE UPPER BASIN

By Thomas C. Adams, Assoc. M. Am. Soc. C.E.

Facts pertaining to the prospective use of the water in the basin above Lee's Ferry, in Arizona.

FIELD TESTS OF A SHALE FOUNDATION

By August E. Niederhoff, Assoc. M. Am. Soc. C.E.

Verification of current theories in soil mechanics by tests on the foundation of Possum Kingdom Dam, on the Brazos River near Mineral Wells, Tex.

THE UNIT HYDROGRAPH PRINCIPLE APPLIED TO SMALL WATERSHEDS

By E. F. Brater, Esq.

Affords a clearer understanding of the natural phenomena involved in the production of a unit hydrograph and indicates that the method is one of the best practical devices for predicting flood flows.

Concern for Society Membership

THE following recommendations of the Society's Committee on Professional Objectives, as reported to the Board of Direction at its July 24, 1939, meeting were approved by the Board:

1. That the Society, through its Local Sections, make a determined and conscious effort to *retain* its present members, and to *obtain* new members. To accomplish this, each Local Committee on Professional Objectives, or a Local Committee on New Membership, or another committee appointed for the purpose, shall be charged with the duty by the Boards of Directors of each Local Section;
2. That Society Headquarters advise each such constituted local committee the names of members in that Section who are about to resign, or to be dropped for non-payment of dues or because of reaching the Constitutional age limit, so that the committee, through appropriate personal contacts with such members, may assist and encourage the members to retain their connection with the Society;

3. That local committees encourage heads of engineering departments to let it be known to their staffs that there is mutual advantage in holding membership in the Society;

4. That each Society member act as a missionary to encourage new applications to the Society, by inviting qualified non-members to attend Section meetings and in other dignified ways;

5. That such local committees take appropriate steps to encourage Juniors and Associates to transfer to higher grades of membership as soon as qualified; and

6. That members of Student Chapters be encouraged by local committees to apply for Junior membership in the Society on graduation.

If followed, these recommendations involve a responsibility on each individual member, as well as on various Society units. They represent the considered thought and desired action as suggested by those who have given most intensive study to one of the great problems of an engineering organization.

News of Local Sections

Scheduled Meetings

COLORADO SECTION—Dinner meeting at the University Club on September 11, at 6:30 p.m.

DAYTON SECTION—Luncheon meeting at the Engineers Club on September 18, at 12:15 p.m.

GEORGIA SECTION—Luncheon meeting at the Atlanta Athletic Club on September 11, at 12:30 p.m. The Fourth Regional Conference on Surveying and Mapping, of which the Section is a joint sponsor, will be held at the Georgia School of Technology on September 6, 7, and 8.

NORTH CAROLINA SECTION—Meeting with the Charlotte Engineers Club in Charlotte on September 18, at 9:30 a.m.

NORTHWESTERN SECTION—Dinner meeting on September 4, at 6:30 p.m.

PITTSBURGH SECTION—Meeting with the Engineers Society of Western Pennsylvania on September 26, at 8:15 p.m.

SACRAMENTO SECTION—Regular luncheon meetings at the Elks Club every Tuesday at 12:10 p.m.

ST. LOUIS SECTION—Luncheon meeting at the Mayfair Hotel on September 24, at 12:15 p.m.

TENNESSEE VALLEY SECTION—Dinner meeting of the Chattanooga Sub-Section at the Y.W.C.A. on September 19, at 6:30 p.m.

Recent Activities

GEORGIA SECTION—*Atlanta, July 13:* A trip through the plant of the Atlantic Steel Company took the place of the regular July meeting of the Georgia Section. Robert W. Sandlin, chief metallurgist of the company, and Earl Shepherd conducted those present through the plant. The group then witnessed a baseball game, and later in the evening a chicken dinner was served in the restaurant of the Atlantic Steel Company. Following dinner, Mr. Sandlin addressed the gathering on the subject, "Steel and the Engineer," discussing particularly the chemical problems involved in making steel.

HAWAII SECTION—*Honolulu, July 11:* The speaker and guest of honor at this dinner meeting was William A. Dittmer, member of a Special Committee on Depreciation of the National Association of Utility Commissions. Mr. Dittmer, who is in Hawaii on leave from the Illinois Commerce Commission, gave an interesting talk on the subject of depreciation.

INDIANA SECTION—*Lafayette, July 16:* This was the occasion of the annual outing held at the Ross Camp of the Purdue University school of civil engineering. Prof. George E. Lommel, who is in charge of the camp, was host at a fine dinner. The list of speakers included J. Emmett Hall, vice-president of the Section, who explained the workings and purpose of the Society; President Elliott, of Purdue University, who outlined briefly his plans for the camp; and David Ross, president of the board of trustees of Purdue. L. E. Martin was then introduced as master of ceremonies and conducted a series of amusing contests.

MICHIGAN SECTION—*East Lansing, May 26:* Joint dinner meeting with the Michigan State College Student Chapter. Carl Hauss, retiring president of the Student Chapter, presided and introduced a number of guests to the membership. Dean Dirke, of the Michigan State College engineering school, then spoke briefly. He was followed by Louis E. Ayres, Director of the Society, who discussed the problems of the Society—particularly those of a financial nature. The principal speaker of the evening was Orion Ulrey, professor of economics at the college. Professor Ulrey's talk on the Scandinavian countries, in which he has traveled and studied extensively, proved of exceptional interest.

OKLAHOMA SECTION—*July 15:* An inspection trip to the site of the Great Salt Plains Dam attracted the largest attendance of any activity of the Section held in recent years. During the afternoon members of the U. S. Engineer Office acted as guides for inspection of the construction of the earth dam and of the excavation of the pits. The methods of making soil tests and the interpretation of the results were thoroughly explained in the soils laboratory. After a dinner in the "mess hall," C. H. Guernsey discussed the history of the Great Salt Plains and the plans under way to establish a wild life refuge there. A talk by W. L. Kuehnle, acting principal engineer in the U. S. Engineer Office at Tulsa, concluded the evening program. Mr. Kuehnle explained the organization of the U. S. Engineer Department and discussed how engineering projects are initiated.

OREGON SECTION—*Portland, July 21:* This meeting was held in honor of Donald H. Sawyer, President of the Society, who was the principal speaker. Colonel Sawyer discussed various phases of Society activity, including its financial status and the need for new members. It was moved, seconded, and unanimously passed that the Section endorse the action of the Board of Direction in recommending Glenn L. Parker to serve the unexpired term of the late Ross K. Tiffany.

SACRAMENTO SECTION—*July 11, 18,* and *25:* Interesting programs were arranged for the regular weekly luncheon meetings of the Section. The speaker at the first of these was H. H. Jaqueth, planning engineer for the city of Sacramento, who discussed the newly established City of Sacramento Housing Authority. On the 18th and 25th the speakers were, respectively, Clifford Wahl, former construction superintendent for the United Fruit Company, and Arthur A. Sauer, office structural engineer in the Division of Architecture of the State Department of Public Works. The latter discussed the organization and work of the Division of Architecture, while Mr. Wahl's subject was "Civilization in Tropical America." *Junior Forum, July 12:* Entertainment on this occasion consisted of the presentation of a film showing the continuous strip process of sheet steel rolling.

SOUTH CAROLINA SECTION—*Greenville, July 14* and *15:* Annual summer meeting of the Section held jointly with a meeting of the South Carolina Society of Engineers. During the technical session several speakers were heard. These included F. H. Calhoun, dean of the school of chemistry and geology at Clemson College, and J. P. Lucas, of the Duke Power Company. Members of the Section also enjoyed a field trip to the Greenville water supply reservoir at Table Rock.

TENNESSEE VALLEY SECTION—*Knoxville Sub-Section, July 11:* The principal speaker at this regular monthly dinner meeting was William C. Fitts, solicitor for the Tennessee Valley Authority and acting head of the TVA legal department. Mr. Fitts' talk on the subject, "Legal Basis for Federal Water Control Project," aroused much discussion. The session concluded with a report on a recent meeting of the Committee on Professional Objectives. This was given by H. L. Fruend, a member of the committee.

ITEMS OF INTEREST

Engineering Events in Brief

The Smeatonian Society of Civil Engineers

NEARLY a hundred and seventy years ago a body of civil engineers was established in England which still continues to hold regular meetings. The creation of John Smeaton, it came together for the first time on March 15, 1771, at the King's Head Tavern, in Holborn, London.

A facsimile copy of the original minute book was recently presented to the Engineering Societies Library. It is a most interesting document, and throws light on the lives of many of the men whose work in the closing years of the eighteenth century was producing fundamental changes in transportation and industry in Britain. The following comments on the the history of this organization, with excerpts from the early minutes, are from an unsigned article on the April 1939 issue of *The Engineering Journal*, monthly publication of the Engineering Institute of Canada:

"By the middle of the eighteenth century, a notable class of civilian engineers had arisen, whose occupation it was to design and construct canals, roads, bridges, harbours, mills and machinery and other works analogous to those formerly executed by engineers in military service.

"John Smeaton, then prominent in engineering work of this kind in England, particularly in connection with the drainage of the fens and the building of the first Eddystone lighthouse, was among the first of these practitioners who adopted the title of 'civil engineer' to distinguish themselves from the military branch. At the same time he saw the advantage to the members of this new profession that would arise if they met together from time to time for friendly discussion. A society was accordingly formed for this purpose; its first meeting took place on March 15th, 1771, in London, at the King's Head Tavern, Holborn, Mr. Smeaton being present although not in the chair. Its first resolution was as follows:

" 'Agreed that the Civil Engineers of this Kingdom do form themselves into a Society consisting of a President, a Vice President, Treasurer Secretary, and other Members who shall meet once a fortnight on Saturday evenings at seven o'clock from Christmas or so soon as any of the country members come to Town (of which they shall give notice to the President) to the end of the sitting of the Parliament at the King's Head in Holborn. It is further agreed that each Member in Town shall pay a forfeit of one shilling for being Absent, unless he is out of Town. It is also agreed that Rules and Orders shall be constituted for the well governing of this Society.'

"Members of the new Society were not long in realizing the fundamental importance of finance in their operations, for at the very next meeting it was found that funds other than the 'forfeits' must be provided and properly administered. Accordingly, it was then voted as follows:

" 'And whereas there will be some expences attending the carrying on the business of the Society it is agreed that all forfeitures shall be applied to that purpose, and that each Member shall pay into the common Stock three pence Weekly to be laid out as the Society shall think proper . . . .'

"This system of finance seems to have proved somewhat difficult in practice, for just a year later it was

" 'Resolved that every person admitted a member shall upon such admission pay one Shilling for the benefit of the Society in lieu of all Fines, Forfeitures and Weekly Contributions.'

"But after only six years, on March 14th, 1777, the amount had to be increased to five shillings.

"These rulings, however, were subject to temporary modification in cases of emergency, for it is recorded that at the very next meeting,

" 'Mr. Pierce our Landlord was elected a Member of this Society and instead of the customary fee for introduction, furnished a five shilling Bowl of Punch and then this meeting was adjourned to this day s'ennight.'

"With such prudent management, and with the motto, *Omnia in numero pondere et mensura*, the Society flourished for twenty years, when its roll of over sixty members included the names of such prominent engineers, natural philosophers, and constructors as Smeaton, Grundy, Whitworth, Priestly, Boulton, Watt, and Rennie.

"Apparently many of the first members were concerned with schemes for public works or enterprises for which parliamentary sanction was needed. This would explain their presence in London and the holding of meetings while Parliament was sitting. No doubt it was this common interest which led to their association as a society for the communication of ideas and knowledge and for regular meetings. The old minute book records that one of these evenings was 'spent canallically, hydraulically, mathematically, philosophically, mechanically, naturally, and socially'; another meeting is chronicled as being 'agreeably contradictory,' a characteristic which is not unknown in some societies of the present day.

"But the young society had its ups and downs. There are apt to be very critical people in all such bodies. Even Smeaton is believed to have commented unfavorably on some of the reports made by brother engineers. There were finally 'some untoward circumstances in the behaviour of one gentleman, towards Mr. Smeaton' which 'gave rise to disunion.' In fact in 1791 there was a distinct unpleasantness. An attempt to patch it up resulted in Mr. Smeaton's accepting an apology, but 'the remembrance of it had an effect on all present.' Shortly thereafter in May, 1792, the society ceased to exist, by mutual consent of its members.

"Without much delay it was arranged to renew the society 'in a better and more respectable form.' Smeaton agreed to be a member, but he died in October of that year, and the first meeting of the reconstituted society did not take place until April, 1793.

"That Society still exists in London as the Smeatonian Society of Civil Engineers. It now consists of forty-eight engineer members and twelve honorary members, who dine together six times a year, and of course have an Annual General Meeting. As Sir Alexander Gibb points out in his life of Telford, the reformed Society was 'from the outset eclectic, and only the senior and the eminent were eligible' for membership. Its list of presidents—continuous since 1841, except for the years of the war—reads like a catalogue of distinguished engineers, including as it does such names as Rennie, the Stephensons, Brunel, Donkin, Siemens, Wolfe Barry, Baker, Kennedy, Unwin, Fitzmaurice and Denny.

"The Society takes pride in maintaining that respect for custom and tradition which is characteristic of so many British organizations. In fact, since 1794, the following toasts have invariably been proposed at its dinners:

" 'The King and Constitution,'
'The Society of Civil Engineers,'
'Absent Members,'
'The Memory of our late worthy brothers: Mr. Smeaton, Mr. Mylne, Mr. Watt and Mr. Rennie,' and
'Success to Waterworks, public or private, that contribute to the comfort, or the happiness of mankind.'

"These are always followed by a sentiment: 'Dam the Canals, Sink the Coal Pits, Blast the Minerals, Consume the Manufactures and Disperse the Commerce of Great Britain and Ireland.'

"At the 150th anniversary of the foundation of the Society, celebrated in 1921, its President, John Strain, M. Inst. C.E., stated that 'forty-seven years after the Society was formed, it constituted a nucleus from which The Institution of Civil Engineers was established on January 2nd, 1818.' Thus the Smeatonian Society of Civil Engineers—probably the oldest engineering society in the world—was in a sense the parent of that senior Engineering Institution whose long and honourable record goes back to the very early days of the development of engineering as a profession."

CIVIL ENGINEERING for October

NATIONALIZATION of the railroads—"a desperate remedy for a desperate disease"—is suggested by Col. William J. Wilgus in an article now scheduled for the October issue of CIVIL ENGINEERING. The author sees no feasible alternative that will avert ultimate disaster. His proposals for action are concrete and detailed.

A concise and fascinating history of surveying, up to the invention of the theodolite in 1571, is the contribution of Prof. W. H. Rayner to the October issue. It is to be followed in November by a second article, tracing the developments of more recent times. The dioptra, the groma, the chorobates, the compass, and the instruments of Agricola are all described and illustrated, and there is a select bibliography for those wishing to pursue the subject further.

Progress in earthquake resistant design is reviewed by Henry D. Dewell in a paper which had its origin on the Annual Convention program. He divides the comparatively short history of seismology and seismic design into four periods, beginning with 1880; the first ends with the San Francisco quake of 1906, the second with the Tokyo and Santa Barbara quakes of the 1920's, the third with the Long Beach quake of 1933, and the fourth continues through 1939. Progress in each period is analyzed, and the paper concludes with a summary of "what we think we know" about earthquake-resistant construction today.

For hydraulic engineers the technique of an interesting piece of field research is described by F. A. McCaughan. It is a study of the value of n in the Trinity River floodway at Dallas. Also for hydraulic and irrigation engineers is the article by M. J. Dowd, deferred from this month's issue, on silt problems in the Imperial Valley.

One paper each on highways, bridges, slum clearance, and public buildings are also on the tentative schedule.

Unwin Biography Published

"THE LIFE and Work of William Cawthorne Unwin" is the title of a 240-page illustrated volume just published by the Unwin Memorial Committee in London. Entire proceeds from sales of this book in the United States will be added to the memorial fund to which numerous American engineers have already contributed.

The life of the late Dr. Unwin, Hon. M. Am. Soc. C.E., says the publisher's announcement, "spanned practically the whole development of modern technical science. [He was born in 1838 and died but six years ago.] His earliest work was with men who are now reckoned among the pioneers of engineering. It is a long step backwards to the times of the construction of the Britannia and Conway and the other wrought-iron bridges of the pre-steel period, to the investigations of Joule, Regnault, and Rankine and the first foundations of thermodynamics, to the earliest uses of iron plates for ship construction. The story of Unwin's life begins at that period and carries the reader through three-quarters of a century of development of civil and mechanical engineering up to the last great civil engineering work upon which his opinion was asked, the Mersey Tunnel.

"To all who have an interest in engineering progress, to the older engineers who knew of Unwin and his works in the days of his zenith, to younger engineers who may wish to learn of the practice of engineering and the development of engineering education in the days of their fathers and even of their grandfathers, this account of Unwin's life should be of interest."

Copies of the biography can be purchased at 12s. 6d. (about $3) each, postpaid, from G. A. Hicks, Unwin Memorial Committee, 15A, Grosvenor Mansions, 82, Victoria Street, London, S.W.1. The memorial fund is to be used to endow an "Unwin Scholarship" at the City and Guilds College, South Kensington, where so much of Unwin's most important work was done.

New Standard for Dimensions of Pipe and Tubing

THE American Standards Association recently announced approval of a new standard for Dimensions and Materials of Wrought-Iron and Wrought-Steel Pipe and Tubing which should result in considerable savings in cost and convenience to manufacturer and user industries. The new standard consists of ten weight and thickness schedules running from the lightest to the heaviest pipe commonly used. Provisions for new materials and more severe service conditions than contemplated, even at present, are provided for in the new schedules in that the thicknesses are not tied to particular pressure-temperature conditions or materials as formerly.

Every effort has been made to coordinate the new with what has gone before. Standard weight and extra-strong thicknesses of the old schedules coincide closely with two of the new schedules. Thus old thicknesses can still be ordered while taking advantage of the new, more elastic schedules.

The standard is available from the American Standards Association, 29 West 39th Street, New York, N.Y., at 50 cents a copy.

Brief Notes from Here and There

INITIATION of a broad program on the coordination of dimensions of building materials and equipment is announced in the August issue of *Industrial Standardization*, house organ of the American Standards Association. The major objective is to promote economies in field erection, but numerous other benefits are expected to be achieved as well. Thirty-six industrial and technical organizations and government groups interested in building materials or housing hold membership on the committee that is to conduct the work. The American Society of Civil Engineers is represented by R. L. Bertin.

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CELEBRATING the centenary of the official founding of New Zealand, an international exhibition will be held in Wellington from November 1939 through April 1940. In connection therewith, the New Zealand Institution of Engineers announces an Engineering Congress, scheduled for February 1940. A cordial invitation to attend is extended to all members of the Society who may be able to do so. Robert B. Brooks has been appointed to represent the Society at the Engineering Congress.

* * * *

"CONSTRUCTION DAY" at the New York World's Fair will be Tuesday, October 3. A program on the theme of "Building Progress in the World of Tomorrow," to be given in the morning of that day in the Hall of Special Events, is scheduled as the main feature of the program. Plans are being sponsored by the New York Building Congress, Inc. Daniel T. Webster, M. Am. Soc. C.E., and George T. Seabury, Secretary, are members of the committee on arrangements.

NEWS OF ENGINEERS

Personal Items About Society Members

HARLOWE M. STAFFORD, previously with the Division of Water Resources, State Department of Public Works, Sacramento, Calif., is now counselor, Region 7, National Resources Planning Board, and engineer in charge of the Pecos River Joint Investigation, New Mexico and Texas. His headquarters are at Roswell, N.Mex.

CHARLES H. YOUNG, president of Young and Stanley, Inc., of Muscatine, Iowa, has sold the firm to the Stanley Engineering Company. Mr. Young will practice independently as a consulting engineer in Muscatine.

WILLIAM WILSON, architect and consulting engineer of New York City, replaces ALFRED RHEINSTEIN as head of the New York City Department of Housing and Buildings. Mr. Wilson was previously adviser to the State Board of Housing on low rent and slum clearance projects and a consultant to the Triborough Bridge Authority. Mr. Rheinstein will now devote all his time to his position as head of the New York City Housing Authority.

JOHN P. HOGAN has resigned as chief engineer of the New York World's Fair to resume private practice as a member of the engineering firm of Parsons, Klapp, Brinckerhoff and Douglas, New York City. He will continue as a vice-president of the

Fair corporation and as its chief engineering consultant.

HARRY H. BAGLEY, until recently with the Turner Construction Company of New York, is now a lieutenant (jg) in the Civil Engineer Corps of the U. S. Navy Department. He is stationed at the Bureau of Yards and Docks, Washington, D.C.

R. A. DAVIDSON is now district bridge engineer for the Pennsylvania Department of Highways, Scranton, Pa. He was formerly an engineer for the Porterfield-Binger Construction Company, Youngstown, Ohio.

NEAL D. SMITH has just been appointed city manager of Ontario, Calif. For the past eleven years he was with the Metropolitan Water District of Southern California in the capacity of office engineer and assistant engineer of construction on the Eagle Mountain and Hayfield pumping plants of the Colorado River Aqueduct.

GEORGE E. LOMMEL succeeds Virgil M. Simmons as chairman of the Indiana State Planning Board. Mr. Lommel is associate professor of civil engineering at Purdue University.

JOHN S. DODDS, professor of civil engineering at Iowa State College, and L. M. MARTIN, who is with the Iowa State Highway Commission, have been reappointed to the Iowa Board of Engineering Examiners. Another appointment to the Board is HOWARD R. GREEN who succeeds CLARE H. CURRIE.

KENNETH K. KING is now located at Indianapolis, Ind., in the capacity of district engineer for District III and utilities engineer for all districts of the Farm Security Administration, U.S. Department of Agriculture. Until lately he was principal sanitary engineer of the Administration in Washington, D.C.

SAMUEL LIPTON, has accepted a position as assistant engineering aid in the National Park Service, Zion National Park, Utah. Formerly he was engineer's helper in the Wyoming State Highway Department at Cheyenne.

ELMER K. NELSON, formerly city engineer of Laramie, Wyo., is now engineer for the state of Wyoming in the tri-state North Platte river water suit.

A. N. CARTER, until lately a civil engineer with the Firestone Plantations Company, in Liberia, has accepted a position as news editor for the *Engineering News-Record*.

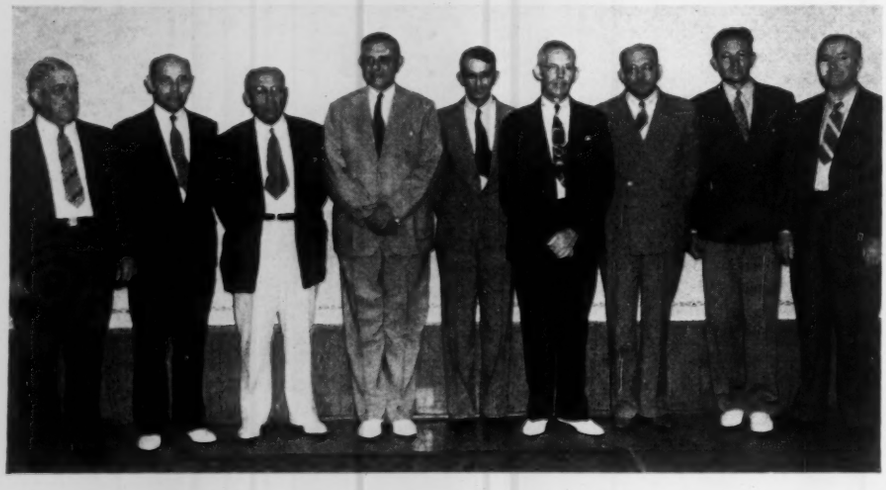
HAROLD E. WESSMAN, professor of structural engineering at New York University for the past two years, has just been appointed chairman of the department of civil engineering in the college of engineering at the university.

WILLIAM P. HUGHES, city engineer of Lewiston, Idaho, has been appointed chairman of the newly created Idaho Engineering Board. Other members of the Society on the Board are R. J. NEWELL, resident engineer for the U. S. Bureau of Reclamation at Boise, who is vice-chairman, and J. E. BUCHANAN, dean of the engineering school at the University of Idaho.

JAMES MACGREGOR MACMARTIN, for half a century with the Delaware and Hudson Railroad, has retired from his position as chief engineer.

ABEL WOLMAN, head of the Maryland State Planning Commission and a member of the faculty at Johns Hopkins University, and W. WATTERS PAGON, consulting engineer of Baltimore, Md., are among the appointees to the new Baltimore City Plan Commission.

WHEN a board of nine engineers met recently on the Lower Rio Grande Project to consider the development of international irrigation facilities for the Lower Rio Grande, it turned out that all were members of the Society. In the accompanying group photo they are arranged from left to right, as follows: James L. Lytel, project engineer for the International Boundary Commission; Alfred Tamm and E. N. Noyes, consulting engineers; L. M. Lawson, American Commissioner for the International Boundary Commission; J. G. White, consulting electrical engineer; Joseph L. Burkholder, senior engineer for the Commission; Royce Tipton, consulting engineer; Richard Stephens, engineer; and R. J. Cummins, consulting engineer.



SOCIETY MEMBERS CONFER ON LOWER RIO GRANDE PROJECT

ELTON D. WALKER, for the past thirty-nine years a member of the faculty at Pennsylvania State College, has retired as the active head of the department of civil engineering. Professor Walker is president emeritus of the Pennsylvania Sewage Works Association, which he was instrumental in organizing, and on June 27 was guest speaker at the annual dinner of this group.

CHARLES P. GROSS, lieutenant-colonel, Corps of Engineers, U. S. Army, has been made district engineer at Rock Island, Ill. He was formerly stationed at Fort Belvoir, Va.

H. J. KRUEGER has been transferred from the New York office of the United States Steel Products Company to Shanghai, China, where he is now office manager. His address is 17 Canton Road.

M. C. BOYER, who is on the staff of the U. S. Geological Survey, has been promoted from the position of junior engineer to that of associate engineer. He is located in Washington, D.C.

WILLIAM T. COLLINS recently resigned as junior assistant engineer for the Muskingum Watershed Conservancy District in order to accept an appointment as junior engineer with the Hydraulic Reports Section of the Huntington District of the U. S. Engineer Corps, with headquarters at Huntington, W.Va.

LEWIS E. MOORE has returned to his former offices at 73 Tremont Street, Boston, Mass., and has resumed practice as an independent consulting engineer. For a time recently he was connected with the engineering firm of Moore and Haller, Inc., of Cambridge, Mass., which is being dissolved.

WILLIAM B. HANLON has left his position as associate engineer with the U. S. Geological Survey to become an engineer with the Medusa Portland Cement Company, with headquarters at 1000 Midland Building, Cleveland, Ohio.

GEORGE L. HALL, acting chief engineer of the Maryland State Department of Health, has been appointed chief of the department, succeeding ABEL WOLMAN, who resigned as chief several months ago. Mr. Hall has been connected with the department for 20 years.

ARMOUR T. GRANGER has been appointed to the position of associate professor of civil engineering at the University of Tennessee, where he will be in charge of instruction in concrete and steel design. For the past ten years Mr. Granger has been assistant engineer in the New York office of Ash-Howard-Needles and Tammen.

CHRISTOPHER E. SHERMAN recently resigned as professor of civil engineering at Ohio State University after 43 years in the civil engineering department.

CLYDE N. BATES, formerly sales engineer for the Republic Portland Cement Company, of San Antonio, Tex., has been appointed assistant sales manager for the Gulf Portland Cement Company, of Houston, Tex.

DECEASED

WILLIAM JOHN ASH (M. '24) engineer of design for the American Cyanamid Company, of New York, N.Y., died on July 10, 1939, at the age of 54. From 1907 to 1914 Mr. Ash was with the Bureau of Engineering at New Brighton, N.Y., and from the latter year until 1917 with the Public Service Commission of New York City. Later he was with the Central Dyestuff and Chemical Company, of Newark, N.J., and with M. Shapiro and Son, of New York City. He joined the staff of the American Cyanamid Company in 1922.

JAMES C. BELL (M. '27) senior construction engineer for the Board of Education at Rochester, N.Y., died on June 10, 1939, at the age of 54. From 1906 to 1924 Mr. Bell was in the Department of the State Engineer and Surveyor of New York, beginning in a subprofessional capacity and later having charge of various construction projects in upstate New York. From 1924 to 1926 he was commissioner of public works of Schenectady, N.Y., and in the latter year became connected with the Board of Education at Rochester.

JOHN MORTON BILLING (Assoc. M. '32) executive engineer for the Public Works Department, Segamat, North Johore, Federated Malay States, died on January 9, 1939. Mr. Billing, who was 35, was born and educated in England. From 1924 to 1926 he was junior assistant in the County Borough Engineer's Office, Corporation of Southend-on-Sea, England. In the latter year he went to Singapore as assistant engineer in the Public Works Department of the Federated Malay States.

JOSEPH PATRICK BURNS (Assoc. M. '08) engineer for Burns Brothers and Haley, of Watertown, N.Y., died on July 4, 1939. He was 64. From 1904 to 1906 Mr. Burns was in the Department of the New York State Engineer, and from the latter year until 1907 a member of the firm, Burns Brothers and Haley, of Watertown. He then served as assistant engineer in the Department of the New York State Engineer for awhile before making a permanent connection with Burns Brothers and Haley.

DANIEL YOUNG DIMON (M. '08) assistant engineer-designer for the New York City Board of Water Supply, died in the Long Island College Hospital on July 8, 1939, at the age of 65. At the outset of his career—in 1896—Mr. Dimon became connected with the American Bridge Company. Later he was with the Foster Wheeler Corporation and, for a number of years, was assistant steel engineer for the George A. Fuller Construction Company. From 1930 to 1938 he was with Gibbs and Hill, and in the latter year he joined the New York City Board of Water Supply. During the war he served as engineer in charge of design and construction in a shipbuilding yard.

GUY CARLETON EMERSON (M. '04) of Boston, Mass., died in that city on July 17, 1939, at the age of 72. During his early career Mr. Emerson was engaged in sewage, subway, and bridge work in Boston, and from 1905 to 1908 he was on irrigation projects for the federal government in Wyoming. He then became consulting engineer for the Boston Finance Commission, where he remained for twenty-five years, retiring in 1936.

FRANCIS JAMES FITZPATRICK (Assoc. M. '13) major, Corps of Engineers, U. S. Army, was killed in an automobile accident on July 15, 1939. Major Fitzpatrick, who was 55, was executive officer for the U. S. Engineer Office at Pittsburgh at the time of his death. Earlier in his career he was employed by various railways, including the Chicago, Milwaukee and St. Paul, the Northwestern Elevated, and the Oregon-Washington Railroad and Navigation Company. In 1911 he became connected with the Isthmian Canal Commission.

LOUIS PREVOST GASTON (M. '05) of Somerville, N.J., died on July 14, 1939, at the age of 74. In 1889 he began his career, working for the Cofrode and Evans Company. Later he was associated, successively, with the Norfolk and Western Railway, the New York, Susquehanna and Western Railroad, and the Raritan Water Power Company. In 1901 he founded the contracting firm of Richards and Gaston, of which he was president from 1910 until his retirement in 1925.

RALPH J. REED (M. '20) former Director of the Society, died in Los Angeles, Calif., on July 27, 1939, at the age of 56. Following his graduation from the University of Michigan in 1908, Mr. Reed entered the employ of the Union Oil Company of California, where he remained until the close of 1929. During the earlier years of this period he served as an engineer in the transportation and sales departments, and from 1918 to 1929 as chief engineer in charge of all engineering design and construction and as executive head of its safety organization. From 1929 on he maintained a private practice in Los Angeles. Mr. Reed was active in the technical affairs of the American Petroleum Institute, particularly in matters of standardization, fire protection, and corrosion study. He also served for three years as a member of its General Standardization Committee and as national chairman of its Committee on Steel Tanks for Oil Storage. Mr. Reed was a former president of the Los Angeles Section of the Society and of the California State Board of Registration for Civil Engineers. Long active in the affairs of the Society, he served as Director from 1933 to 1935.

RALPH J. REED

CECIL NEIL HAGGART (Assoc. M. '08) structural engineer of Pittsburgh, Pa., died on July 29, 1939, at the age of 64. Early in his career Mr. Haggart worked for both the American Bridge Company and the McClintic-Marshall Company, and from 1908 to 1912 he was bridge designer for the New York State Barge Canal. He then established the private structural practice which, except for a two-year period, he maintained until his death.

RALPH STEVENSON HAWLEY (M. '11) city engineer of Emeryville, Calif., died on July 30, 1939, at the age of 62. From 1903 to 1907 Mr. Hawley was with the U. S. Geological Survey, and from 1907 to 1908 in the city engineer's office at Oakland, Calif. In the latter year he became city engineer of Emeryville, remaining in this position until his death.

WILLIAM JOSEPH JUDGE (Assoc. M. '34) retired hydraulic and dredging engineer of Cooperstown, N.Y., died on June 30, 1939, at the age of 59. Colonel Judge was engaged on many projects, including the construction of parts of the Panama Canal, reclamation work for the British Government in Trinidad, and construction work in Greece for the Monks-Ulen Company of New York. He was a veteran of the Spanish-American War and of the World War, during which he commanded the 303d Engineers, A.E.F., in France.

DAVID WILLIAM MORROW (M. '19) retired consulting engineer of Cleveland, Ohio, died there on July 5, 1939, at the age of 72. From 1893 to 1901 Mr. Morrow was with the Variety Iron Works Company, and from the latter year to 1911 with the Cleveland Railway Company. As head of his own consulting practice, established in 1911, Mr. Morrow designed a reservoir for the city of Detroit and numerous water works and reservoirs in Ohio. He retired several years ago.

ERI OLMSTEAD (Assoc. M. '37) traveling engineer for the PWA at Highland Park, Mich., died on April 13, 1939, at the age of 53. Mr. Olmstead was with the Stone and Webster Engineering Corporation from 1912 to 1916, and the Harry M. Hope Engineering Corporation from 1916 to 1920. In the latter year he was made superintendent of construction for J. A. Utley, contracting engineer of Detroit, Mich., remaining there until 1925 when he became president of the general contracting firm, Beyster and Olmstead. Mr. Olmstead joined the staff of the PWA in 1934.

JEAN LECLERC DE PULLIGNY (M. '12) inspecteur général des ponts et chaussées en retraite, Antibes, France, died on April 7, 1939. Commandant de Pulligny, who was 88, spent his entire professional career in his native country, France. Though ad-

vanced in age at the time of the war, he volunteered as a major of military engineers, which grade he had formerly occupied as an officer in the reserve army, and was put in charge of a road builders' group of the 8th army on the Lorraine front. Later he was inspector general to the ministry of munitions and, in 1919, was decorated for his services with the French military cross. He was made inspector general of bridges and roads in 1920.

JOHN MATTHEW RAYBURN (M. '17) civil and mining engineer of Pittsburgh, Pa., died at his home in that city on July 20, 1939. Mr. Rayburn, who was 69, spent the early years of his career in mining, railroad, and irrigation work for various firms in this country and Mexico. From 1904 on he was engaged in the general practice of engineering in Pittsburgh, specializing in shaft design and construction and coal-mine projections.

BENJAMIN BRUCE SHAW (M. '22) since 1934 manager of the Georgia Kaolin Company, Drybranch, Ga., died on June 2, 1939, at the age of 53. Mr. Shaw was with the Chicago, Rock Island and Pacific Railway Company from 1913 to 1922, and was chief engineer of the Cuba Railroad from the latter year until 1925. He then became sales engineer for the Argyle Railway Supply Company, Chicago, and later was assistant state engineer for the PWA in Illinois.

LOUIS ALOYSIUS SLATTERY (Affiliate '36) vice-president of the Hudson River Stone Corporation, New York, N.Y., died on February 24, 1939. He was 51. Mr. Slattery had worked for the U. S. Realty and Improvement Company, the Federal Shipbuilding Company, the Frederick Snare Corporation, and the Interborough Rapid Transit Company. From 1925 to 1938 he was secretary and treasurer of the Lyons-Slattery Company, Inc., and coincident with this (from 1931 on) he was vice-president and general manager of the Hudson River Stone Corporation.

WALDO WADE WAGGONER (M. '23) consulting engineer of Nevada City, Calif., died on May 18, 1939, at the age of 78. During the early years of his career Mr. Waggoner was engaged in mining engineering construction. From 1898 to 1906 he was county surveyor of Nevada County, California, and later was engineer, successively, for the River Mines Company, the Eureka Lake and Yuba Canal Company, and the Northern Water and Power Company. He served as state debris commissioner and, at one time, was mayor of Nevada City.

RALPH WATTS WARDWELL (Assoc. M. '14) assistant engineer in the U. S. Engineer Office at Providence, R.I., died at his home in that city on June 28, 1939. Mr. Wardwell, who was 60, had been employed by the Colorado Springs Light, Heat and Power Company, the Atchison, Topeka and Santa Fe Railway, and various other organizations. During the war he served as a captain in the Corps of Engineers, U. S. Army. Later, he maintained a consulting practice, specializing in valuation work.

The Society welcomes additional biographical material to supplement these brief notes and to be available for use in the official memoirs for "Transactions."

WILLIAM TIBBITTS WEBB (Assoc. M. '13) purchasing agent for the Emprezas Electricas Brasileira at Rio de Janeiro, Brazil, died suddenly in that city on June 13, 1939. Mr. Webb, who was 59, got his early experience working for the city engineer of Providence, R.I., and for various railroads. Beginning in 1908, he was for some years engineer for the Quartermaster at the U. S. Military Academy at West Point, N.Y. Later he went to Brazil, and he was connected with the Emprezas Electricas Brasileira for a number of years before his death.

AUBREY WEYMOUTH (M. '11) vice-president and chief engineer of Post and McCord, Inc., of New York, N.Y., died on July 27, 1939, at the age of 66. Mr. Weymouth had been connected with Post and McCord since 1897. He was made chief engineer in 1907 and vice-president in 1930. During his long period of connection with the company it supplied and erected the steel for the Empire State Building, the Chrysler Building, and many other notable structures in New York City.

CHARLES POPE YEATMAN (M. '87) retired civil engineer of Pasadena, Calif., died on June 23, 1939, at the age of 86. Most of Mr. Yeatman's career was spent in railroad work in this country and South America. He first went to South America in 1877 as chief engineer of the Antioquia Railway in Colombia. After seven years in this capacity, he returned to railroad work in the United States, going again to Colombia in 1892. From 1895 to 1911 he was, successively, assistant manager and manager of the Barranquilla Railway and Pier Company in Colombia. In the latter year he retired.

Changes in Membership Grades

Additions, Transfers, Reinstatements, and Resignations

From July 10 to August 9, 1939

ADDITIONS TO MEMBERSHIP

AGRAWAL, PITAM CHAND (M. '39), Executive Engr., Upper Div., Agra Canal, Muttra, United Provinces, India.

BERMAN, SIDNEY (Jun. '39), Subway Rodman, City of Chicago, Dept. of Subway and Traction, 22 East Huron St. (Res., 1442 South Kedzie Ave.), Chicago, Ill.

BURRELL, GENE NATHANIEL (M. '39), Hydr. Engr., TVA, 3004 Fountain Park Boulevard, Knoxville, Tenn.

CALLIGARO, LEO (Jun. '39), With Public Service Gas & Elec. Co. of New Jersey, 200 East 5th St., Paterson (Res., 49 East 8th St., Clifton), N.J.

CHOU, CHEN-LUN (M. '39), Director and Engr.-in-Chf., Canton Pearl River Back Reach Impvt. Bureau of Whampoa Port Development Administration and Canton New Waterworks Constr. Bureau, Canton, China.

CLOUSER, LEVERNE HILL (Assoc. M. '39), San. Engr., TVA, 508 Union Bldg., Knoxville, Tenn.

DARMSTAEDTER-HELVERSEN, FRITZ HANS LUDWIG (Jun. '39), Graduate House, Mass. Inst. Tech., Cambridge, Mass.

DINSMORE, BURTON FREMONT (M. '39), 1011 South Fremont Ave., Alhambra, Calif.

DUNBAR, CRAWFORD KYLE (Assoc. M. '39), Asst. Constr. Engr., RA, Box 2033, Atlanta, Ga.

TOTAL MEMBERSHIP AS OF AUGUST 9, 1939	
Members	5,672
Associate Members	6,443
Corporate Members	12,115
Honorary Members	28
Juniors	3,957
Affiliates	73
Fellows	1
Total	16,174

EMERSON, LEWIS AZRO (M. '39), Prin. Civ. Engr., South Carolina Public Service Authority, Peoples Bldg., Charleston (Res., 1133 Glenwood Rd., Columbia), S.C.

FIESENHEISER, ELMER (Assoc. M. '39), Asst. Highway Bridge Engr., U. S. Bureau of Public Roads, 907 Main Post Office Bldg. (Res., 843 Clear St.), St. Paul, Minn.

FINLEY, GEORGE BUCHANAN (Assoc. M. '39), Dist. Engr., State Highway Dept., Box 471, Corpus Christi, Tex.

FREEMAN, GEORGE DONALD (Jun. '39), Junior Hydr. Engr., U. S. Dept. of Interior, Geological Survey, 338 Security Mutual Bldg., (Res., 27 Mozart St.), Binghamton, N.Y.

GILBERT, ROBERT PIERCE (Jun. '39), With U. S. Engrs., Room 219 Post Office, Stockton, Calif.

CROSSMANN, LOUIS CARL (Assoc. M. '39), Asst. Dist. Engr., Federal Buildings Administration, PWA, 1201 Custom House, Boston, Mass.

HAIT, EDWIN A. (Jun. '39), Junior Eng. Insp., PWA, 2923 East 77th St., Chicago, Ill.

HANSEN, VIGGO (M. '39), Civ. Engr., Philadelphia Elec. Co., 900 Sansom St., Philadelphia (Res., 223 Wembly Rd., Upper Darby), Pa.

HOFFMAN, ROBERT FRANCIS (Jun. '39), Junior Hydr. Engr., TVA, Union Bldg. (Res., 2106 North Broadway), Knoxville, Tenn.

HOFFMEISTER, JOHN FERNARD, JR. (Assoc. M. '39), Acting Chf. Estimator, Dravo Corporation, Neville Island Branch, Pittsburgh (Res., 646 Maple Lane, Sewickley), Pa.

JAMES, CLYDE NOBLE, JR. (Jun. '39), Instrumentman, State Highway Dept., Box 386, Levelland, Tex.

JEFFREY, WILLIAM MAXSON (Assoc. M. '39), Highway Engr., Bureau of Public Roads, 254 Custom House (Res., 49 South Clarkson St.), Denver, Colo.

JOHNSON, ALFRED WALTON (Assoc. M. '39), Senior Engr., Soils, State Highway Comm., 2141 Kansas Ave., Topeka, Kans.

JONES, JAMES EDWARD (Assoc. M. '39), Supt. of Constr., Atlantic Refining Co., 260 South Broad St., Philadelphia (Res., 811 East Darby Rd., Llanerch, Upper Darby P.O.), Pa.

JUSTIN, JOEL BATES (Assoc. M. '39), Plant Supt., Appalachian Elec. Power Co., Box 8, Dublin (Res., 116 Eleventh St., Pulaski), Va.

KUSS, THEODORE MARTIN (Assoc. M. '39), Chf. Engr., Pacific Bridge Co., 333 Kearny St., San Francisco, Calif.

LANDER, FRANK TEMPLETON, JR. (Jun. '39), Junior Hydr. Engr., U. S. Geological Survey, 300 State Highway Bldg., Austin, Tex.

LANGDON, LAWRENCE ELMER (Assoc. M. '39), Secy., Pacific Flush Tank Co., 4241 Ravenswood Ave., Chicago, Ill.

LEVINE, EDWARD (Jun. '39), With Black & Veatch, 4706 Broadway, Kansas City, Mo. (Res., 954 North Parkway Boulevard, Memphis, Tenn.)

LEWIS, TAYLOR DOWNER (Jun. '39), Traffic Engr., Police Dept. (Res., 700 Johnston St.), Gary, Ind.

MACLEOD, JAMES HENRY (Jun. '39), Senior Eng. Aid, Met. Dist. Water Supply Comm., 4 Walker St. (Res., 201 Lakeside Ave.), Marlboro, Mass.

MCJUNKINS, ORREN RAINEY (Assoc. M. '39), Engr., U. S. Engrs., 1068 Navy Bldg., Washington, D.C. (Res., George Mason Hotel, Alexandria, Va.)

MIDDLETON, EWEL VENARD (Assoc. M. '39), Instr. in Civ. Eng., Texas Technological Coll. (Res., 2103 Eighteenth), Lubbock, Tex.

MOORE, ROBERT THOMAS (Assoc. M. '39), Asst. Div. Engr., Great Lakes Dredge & Dock Co., 1100 Morgan Bldg., Buffalo, N.Y.

MULLER, FRED ADOLF (Jun. '39), Junior Engr., Anchor Buildings, Inc., 1433 Thirty-Eighth St. (Res., 60–12 Seventieth Ave.), Brooklyn, N.Y.

MURPHY, GERARD JOSEPH (Jun. '39), 674 Seventy-Eighth St., Brooklyn, N.Y.

NEILSON, EDWIN PAUL (Assoc. M. '39), With Buffalo Sewer Authority (Res., 734 Tacoma Ave.), Buffalo, N.Y.

NORRIE, KENNETH PETER (Jun. '38), 408 Lakeside South, Seattle, Wash.

PERKINS, CALVIN EUGENE (Assoc. M. '39), Asst. Prof., Civ. Eng., Oklahoma Agri. and Mech. Coll., Stillwater, Okla.

PETERKA, ALVIN JOSEPH (Jun. '39), Junior Hydr. Engr., TVA, Box 422, Norris, Tenn.

PHILLIPS, GEORGE DWIGHT (Assoc. M. '39), Constr. Engr., B. Perini & Sons, Inc., Lackawack (Res., Sundown), N.Y.

QUINN, IRVING (Jun. '39), Insp., Ash-Howard-Needles & Tammen, West Wildwood (Res., 120 West Juniper Ave., Wildwood), N.J.

REEDER, KENNETH ABRAHAM (M. '39), Structural Engr., Southern California Edison Co., 601 West 5th St., Los Angeles (Res., 1860 Meadowbrook Rd., Altadena), Calif.

RUNYON, FREDERICK OSCAR (M. '39), Cons. Engr. (Runyon & Carey), 33 Fulton St., Newark (Res., 26 Hickory Drive, Maplewood), N.J.

SAUER, RICHARD PAUL (Jun. '39), Engr., Div. of Grazing, D.G. 120, Winnemucca, Nev.

SEAWELL, LACY WEDDINGTON (Assoc. M. '39), Structural Engr., Luke Seawell, 152 Nassau St., N.W., Atlanta, Ga.

SELBY, GARLAND FERD (Assoc. M. '39), Office Engr., The Road Supply and Metal Co., Topeka, Kans.

SHEEHY, JOHN EDWARD (Jun. '39), Senior Eng. Aid, Bridge Dept., State Div. of Highways, Box 1499, Sacramento (Res., 2634 North McCall, Selma), Calif.

SLOAN, GARRETT (Jun. '39), 180 Lenox Rd., Brooklyn, N.Y.

SMITH, HERBERT EUGENE, JR. (Assoc. M. '39), Asst. Civ. Engr., City of Cleveland, City Hall (Res., 10507 Lake Ave., Apartment 406), Cleveland, Ohio.

SPARLING, RAYMOND CARTER (Jun. '39), Pacific Coast Sales Mgr., R. W. Sparling, 945 North Main, Los Angeles (Res., 941 East Beverly Drive, San Gabriel), Calif.

STATHIS, NICHOLAS PETER (Assoc. M. '39), Asst. Structural Engr., Bureau of Chemistry and Eng., U. S. Dept. of Agriculture (Res., 3905 Davis Pl., N.W.), Washington, D.C.

STEACY, ROBERT EVANS (Jun. '39), 2440 Amsterdam Ave., New York, N.Y.

TOUBY, HARRY (Jun. '39), With the Geo. D. Auchter Co., 2016 East Adams St., Jacksonville, Fla.

TURNBULL, JOHN MACNEIL (Assoc. M. '39), Asst. Engr., Hume Pipe Co. (Australia) Ltd. (Res., 2 Howitt St.), Camberwell S.E.6, Victoria, Australia.

TURNIPSEED, CHARLES (Assoc. M. '39), Asst. Office Bridge Engr., State Highway Comm., Masonic Temple (Res., 2832 Michigan), Topeka, Kans.

VARNEY, FORREST FRANKLIN (Assoc. M. '39), Asst. Engr., U. S. Engrs., 418 Patterson Bldg., Fresno, Calif.

WALKER, HARLEY REGINALD (Jun. '39), Engr., Lubbock Steel Works, Box 1493 (Res., 2017 Fourteenth St.), Lubbock, Tex.

WALTERS, SAMUEL FRANKLIN (Assoc. M. '39), Res. Engr., State Highway Comm., 1345 North East 59th Ave., Portland, Ore.

WILLIAMS, JESSE EUGENE (Assoc. M. '39), Highway Engr., U. S. Bureau of Public Roads, 254 Custom House (Res., 330 South Downing St.), Denver, Colo.

WOLFE, DEWITT CLINTON (Assoc. M. '39), Branch Mgr., Sverdrup & Parcel, 1109 Transamerica Bldg., Los Angeles, Calif.

WORLEY, HERBERT EVERETTE (Jun. '39), Asst. Engr., State Highway Comm., 2141 Kansas Ave., Topeka, Kans.

ZOBEL, WILLIAM ALEXANDER (Assoc. M. '39), Asst. Structural Engr., TVA, Highway and Railroad Div., James Bldg., Chattanooga, Tenn.

MEMBERSHIP TRANSFERS

AMMANN, WERNER (Jun. '30; Assoc. M. '39), Designer, Bethlehem Steel Co., Fabricated Steel Constr., 701 East 3d St. (Res., 1018 Raymond Ave.), Bethlehem, Pa.

BRADLEY, STEPHEN ROWE, JR. (Jun. '31; Assoc. M. '39), Asst. Treas., Central Hudson Gas & Elec. Corporation, South Rd., Poughkeepsie, N.Y.

BUHLER, FRED WILLIAM (Jun. '30; Assoc. M. '39), Surv., L. A. Boyd Arctic Expedition, Am. Geographical Society, Broadway at 156th St., New York (Res., 200–15 One Hundredth Ave., Hollis), N.Y.

BUTLER, GORDON HUBERT, SR. (Assoc. M. '20; M. '39), Pres., Polaris Concrete Products Co., Box 86, West Duluth, Minn.

CLARK, JOHN ALFRED (Jun. '30; Assoc. M. '39), Associate San. Engr., FSA, U. S. Dept. of Agriculture, 85 Second St., Room 401, San Francisco (Res., 2644 Dwight Way, Berkeley), Calif.

EDLUND, LAWRENCE LINNÉ (Assoc. M. '23; M. '39), Engr. of Design, Armour & Co., Union Stock Yards (Res., 8220 Langley Ave.), Chicago, Ill.

FAIRBAIRN, JOHN MORRICE ROGER (M. '13; Hon. M. '39), 1939 St. Luke St., Montreal, Canada.

FLORAS, CHRISTOS LAZARÉ (Jun. '32; Assoc. M. '38), Prof. of San Eng., State School of Hygiene, 5 Lefcossias St., Athens, Greece.

HANAVAN, WILLIAM LAWRENCE (Jun. '04; Assoc. M. '10; M. '39), Asst. Engr. Designer, Board of Water Supply, 346 Broadway, Room 1224, New York, N.Y.

HANSON, THOMAS COOPER (Jun. '36; Assoc. M. '39), Asst. Prof., Civ. Eng., Univ. of Detroit, Detroit, Mich.

HARVEY, ALFRED LAIRD (Assoc. M. '31; M. '39), Asst. Engr., Tyne Impvt. Comm., Engr's. Office, Bewick St., Newcastle upon Tyne 2, England.

HOWE, JOHN EDWARD (Jun. '28; Assoc. M. '39), Architectural and Equipment Draftsman, Eastman Kodak Co., Kodak Park, Rochester (Res., 76 Valley View Rd., Irondequoit), N.Y.

HUNT LOREN WILSON (Jun. '32; Assoc. M. '39), Senior Insp. of Constr., U. S. N., Bureau of Yards and Docks, Naval Air Station, Alameda (Res., 2625 Ridge Rd., Berkeley), Calif.

LARGE, GEORGE ELWYN (Assoc. M. '28; M. '39), Associate Prof., Civ. Eng., Ohio State Univ. (Res., 2201 Yorkshire Rd.), Columbus, Ohio.

LENZ, ARNO THOMAS (Jun. '32; Assoc. M. '39), Asst. Prof., Hydr. and San. Eng., Univ. of Wisconsin, Hydr. Laboratory, Univ. of Wisconsin (Res., Shorewood Hills), Madison, Wis.

LOEHLER, JOHN GUSTAV (Assoc. M. '30; M. '39), Cons. Engr., 726 Jackson Pl., N. W., Washington, D.C.

MAUZY, HARRIS KENNETH (Jun. '32; Assoc. M. '39), Asst. Highway Engr., State Div. of Highways, Bridge Dept., Box 470, Turlock, Calif.

MEADOWCROFT, NIEL FAIRHURST (Jun. '33; Assoc. M. '39), Asst. Hydr. Engr., U. S. Forest Service, Phelan Bldg., San Francisco (Res., 151 Oakdale Ave., Mill Valley), Calif.

MOHR, HENRY ARTHUR (Assoc. M. '23; M. '39), 1865 Beacon St., Wahan, Mass.

RAATZ, WALTER ARTHUR (Jun. '31; Assoc. M. '39), Asst. Engr., Galveston Wharf Co., Galveston, Tex.

RIDGEWAY, GEORGE ALLEN (Assoc. M. '15; M. '39), Senior Engr., State Highway Dept., Jefferson City, Mo.

RILEY, ELMER DARRELL (Jun. '30; Assoc. M. '39), Senior Eng. Aid, State Dept. of Public Works, Div. of Eng., 301 East Water St., Syracuse (Res., Richland), N.Y.

SANDBERG, CLIFFORD HELMER (Assoc. M. '32; M. '39), Asst. Engr., Bridge Dept., A. T. & S. F. Ry., Railway Exchange Bldg. (Res., 7610 Eastlake Terrace), Chicago, Ill.

SEMON, EDWARD GOODRICH (Assoc. M. '20; M. '39), Structural Engr., Gen. Elec. Co., Constr. Eng. Dept., Schenectady, N.Y.

SPERLING, ELMER JOHN (Jun. '31; Assoc. M. '39), With U. S. Engrs. Office, Post Office Bldg. (Res., 4141 West Cullom), Chicago, Ill.

STEPHENS, UEL (Jun. '17; Assoc. M. '21; M. '39), Associate Regional Engr., PWA, 803 Electric Bldg. (Res., 2544 Wabash Ave.), Fort Worth, Tex.

STERN, CHARLES GEORGE (Jun. '31; Assoc. M. '39), Office Engr., Inspection Div., FSA (Res., 1740 Euclid St., N.W., Apartment 7), Washington, D.C.

SWEET, CHARLES LATHAM (Jun. '28; Assoc. M. '39), Res. Engr., State Div. of Highways, Bridge Dept., Box 1499, Sacramento, Calif.

THOMSON, HARRY FREEMAN (Assoc. M. '28; M. '39), Vice-Pres., Gen. Material Co., 605 Buder Bldg., St. Louis, Mo.

VAN LIEW, ROSCOE EMERSON (Jun. '33; Assoc. M. '39), Associate Engr., U. S. Bureau of Reclamation, 339 Federal Bldg. (Res., 1717 South 5th East), Salt Lake City, Utah.

WALDRON, HEBER GRAFENSTEIN (Jun. '34; Assoc. M. '39), Res. Engr., State Highway Dept., Valley City, N.Dak.

REINSTATEMENTS

BELL, MACE HUDSON, JR., Jun., reinstated July 17, 1939.

BOSSEN, LESLIE EDWIN, Jun., reinstated July 10, 1939.

ELAM, EDWIN ESSEX, Assoc. M., reinstated July 21, 1939.

GIDLOW, WILLIAM, Jun., reinstated Aug. 9, 1939.

GOULD, MAURICE AUGUSTUS, Assoc. M., reinstated July 31, 1939.

LAGAARD, MAURICE BERNHART, M., reinstated July 17, 1939.

NORELIUS, LEWIS MAGNUS, M., reinstated Aug. 7, 1939.

ROGERS, ALVA LEWIS, Assoc. M., reinstated Aug. 2, 1939.

SCOTT, DONALD COZINE, Assoc. M., reinstated July 10, 1939.

WOODROUGH, KENDAL ALDRICH, Assoc. M., reinstated July 17, 1939.

RESIGNATIONS

BELSTERLING, RICHARD GORDON, Jun., resigned July 6, 1939.

DUBOIS, GEORGE BACHE, M., resigned Aug. 3, 1939.

JONES, EDWIN ALFRED, Jun., resigned July 7, 1939.

Applications for Admission or Transfer

Condensed Records to Facilitate Comment from Members to Board of Direction

September 1, 1939 NUMBER 9

The Constitution provides that the Board of Direction shall elect or reject all applicants for admission or for transfer. In order to determine justly the eligibility of each candidate, the Board must depend largely upon the membership for information.

Every member is urged, therefore, to scan carefully the list of candidates published each month in CIVIL ENGINEERING *and to furnish the Board with data which may aid in determining the eligibility of any applicant.*

It is especially urged that a definite recommendation as to the proper grading be given in each case, inasmuch as the grading must be based upon the opinions of those who know the applicant personally as well as upon the nature and extent of his professional experience. Any facts derogatory to the personal character or professional reputation of an applicant should be promptly communicated to the Board.

Communications relating to applicants are considered strictly confidential.

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of 30 days, and from non-residents of North America until the expiration of 90 days from the date of this list.

MINIMUM REQUIREMENTS FOR ADMISSION

GRADE	GENERAL REQUIREMENT	AGE	LENGTH OF ACTIVE PRACTICE	RESPONSIBLE CHARGE OF WORK
Member	Qualified to design as well as to direct important work	35 years	12 years	5 years RCM*
Associate Member	Qualified to direct work	27 years	8 years	1 year RCA*
Junior	Qualified for sub-professional work	20 years	4 years	
Affiliate	Qualified by scientific acquirements or practical experience to cooperate with engineers	35 years	12 years	5 years RCM*

* In the following list RCA (responsible charge—Associate Member standard) denotes years of responsible charge of work as principal or subordinate, and RCM (responsible charge—Member standard) denotes years of responsible charge of IMPORTANT work, i. e., work of considerable magnitude or considerable complexity.

APPLYING FOR MEMBER

AINSWORTH, CULVER MARTIN, El Paso, Tex. (Age 50) (Claims RCA 7.0 RCM 14.5) 1927 to date Engr., International Boundary Comm., United States and Mexico, on surveys and construction of boundary projects.

BURNHAM, WALTER CLINTON (Assoc. M.), Oklahoma City, Okla. (Age 51) (Claims RCA 5.8 RCM 18.9) May 1935 to Aug. 1938 Engr., Oklahoma Conservation Comm.; previously with CWA-FERA, Medicine Park, Okla., as Supt. on dam and road work and then Superv. Engr. in 20 southwestern counties.

CARTER, ISAAC NEWTON (Assoc. M.), Moscow, Idaho. (Age 48) (Claims RCA 1.0 RCM 6.3) Sept. 1923 to date with Univ. of Idaho as Student Instructor, Instructor, Asst. Prof. of Civ. Eng., and (since Sept. 1934) Associate Prof. of Civ. Eng.

FLANAGAN, WILLIAM EDWARD, Washington, D.C. (Age 53) (Claims RCA 0.0 RCM 20.5) 1934 to date Engr. Inspector being Senior Engr., PWA, Boston, Mass. (1 year), and Supervising Constr. Engr. in charge of dam construction, plant layout and design, U. S. Govt., Dept. of Agriculture.

FRIEDRICH, LAWRENCE MAX, Toledo, Ohio (Age 38) (Claims RC 7.7 D 6.7) Sept. 1937 to date at Univ. of Toledo as Instructor in, and (at present) Asst. Prof. of Civ. Eng.; previously with W. H. Friedrich, Contr. on construction of concrete roads, pavements, bridges, grade separations, etc.

FROST, LLOYD GARNER (Assoc. M.), New Orleans, La. (Age 45) (Claims RCA 4.0 RCM 8.8) Feb. 1938 to date Director, Louisiana Geodetic Survey; previously Cons. Engr., Monroe, La.; with FERA and WPA as Parish Engr., and Dist. Engr.

GADDY, JOHN WILLIAM, Lake Charles, La. (Age 44) (Claims RCA 0.0 RCM 16.6) March 1939 to date Engr. and Contr.; April to June 1934 and April 1936 to March 1939 estimating, drafting, buying, also Traveling Supt. and Director of Personnel with A. Farnell Blair and H. M. Knapp, Contrs.; in the interim Res. Engr. Inspector, PWA, Arkansas.

HAINSWORTH, BRUCE ALEXANDER, Houston, Tex. (Age 43) (Claims RCA 5.0 RCM 9.6) April 1928 to June 1932 Designing Engr., being Structural Designer, and June 1938 to date Project Engr., The H. K. Ferguson Co., Cleveland, Ohio;. in the interim Chf. Engr., The Sharpless Solvents Corporation, Wyandotte, Mich.

HANNING, WILLIS COSINE (Assoc. M.), Brooklyn, N.Y. (Age 38) (Claims RC 14.6 D 2.7) Jan. 1939 to date Acting Chf. Engr., Borough of Queens, also at various times Acting Borough Supt.; previously Asst. Engr., Bureau of Bldgs. (Dept. of Housing and Bldgs., Div. of Bldgs.), Borough of Manhattan.

HEMMERT, JOHN LESLIE, Sacramento, Calif. (Age 43) (Claims RCA 6.5 RCM 3.8) April 1930 to date with California Div. of Highways as Asst. Physical Testing Engr., Associate Physical Testing Engr., and (since July 1938) Senior Physical Testing Engr.

HODSON, ROBERT WILLIAMS, Kirkwood, Mo. (Age 49) (Claims RCA 4.7 RCM 21.9) Oct. 1921 to date with Missouri State Highway Dept., Jefferson City, Mo., as Project Engr., acting as Res. Engr., and (since July 1922) Div. Engr.

HOLSER, FLOYD LESTER, Los Angeles, Calif. (Age 49) (Claims RCA 4.0 RCM 23.1) Jan. 1939 to date with WPA; Jan. 1921 to Jan. 1939 with F. L. Holser & Co., Engrs. & Contrs., on construction of Union Pacific Freight Depot at Los Angeles, various small buildings and bridges, etc.

JONES, WILBOURNE OCIE (Assoc. M.), Fort Worth, Tex. (Age 42) (Claims RCA 9.3 RCM 10.3) May 1921 to Aug. 1937 First Asst. to City Engr. and Aug. 1937 to date City Engr., Fort Worth.

MILLER, JOSEPH ALBERT (Assoc. M.), New Orleans, La. (Age 35) (Claims RC 12.3 D 12.3) Sept. 1933 to date Structural Engr., Woodward-Wight Co.; previously with Truscon Steel Co.

MORELAND, JAMES EDGAR (Assoc. M.), Chattanooga, Tenn. (Age 47) (Claims RC 22.1 D 8.5) July 1935 to date with TVA, Knoxville and Chattanooga, as Asst. Engr. and Administrative Officer, Materials Dept., Highway Engr., Senior Highway Engr. and Asst. Head, Highway and Railroad Div., and (since June 1939) Prin. Highway Engr.; previously with Tennessee State Highway Dept. as Asst. Engr. (2 years), Div. Engr. (over 7 years), Constr. Engr. (over 1 year), and Engr. of Designs (6 years).

MURRAY, RAY (Assoc. M.), Moylan, Pa. (Age 63). (Claims RCA 18.2 RCM 17.1) July 1934 to date Contr. Engr. on railroad bridge work.

PATERSON, JOHN LAMB (Assoc. M.), Shanghai, China. (Age 38) (Claims RCA 3.0 RCM 8.0) 1934 to date with Palmer & Turner, Archts. & Civ. Engrs., since 1937 as member of firm.

PATTERSON, ARTHUR PRUDEN, Madison, Wis. (Age 45) (Claims RCA 0.0 RCM 17.2) Aug. 1935 to date State Director of Operations, WPA, Federal Govt., Administrative and Technical Head of Div. in charge of all construction in Wisconsin; previously Dist. Engr., Wisconsin ERA.

POWELL, FAY EDWIN (Assoc. M.), Balboa, Canal Zone. (Age 53) (Claims RCA 12.1 RCM 9.5) 1929 to Dec. 1934 Asst. Constr. Q.M., and Dec. 1934 to date Constr. Q.M., the Panama Canal.

SEAWELL, VOLNEY RICHARD, Los Angeles, Calif. (Age 55) (Claims RCA 5.7 RCM 21.9) Oct. 1938 to date with PWA as Res. Engr. Inspector on school buildings, and (since March 1939) Director, Div. of Operations; previously Senior Project Engr., Chf. of Inspection Sec., Zone Engr. and Mgr., Project Control Dept., WPA; private work with E. C. Taylor investigating manufacturing and mining properties; Bldg. Inspector, Board of Education, Los Angeles, Calif.

SMITH, PAUL ALBERT (Assoc. M.), Washington, D.C. (Age 38) (Claims RC 6.2 D 2.2) Aug. 1924 to date with U. S. Coast and Geodetic Survey as Jun. Engr., Observer, Jun. Hydrographic, and Geodetic Engr., Asst. to Chf. of Sec. of field work, and (since May 1938) Hydrographic and Geodetic Engr.

TYLER, IVAN LOUIS (Assoc. M.), Harrisburg, Pa. (Age 41) (Claims RCA 4.8 RCM 7.3) 1939 to date Concrete and Materials Engr., Pennsylvania Turnpike Comm.; previously Material Engr., TVA, Knoxville, Tenn.

VERMILYA, FLOYD DURLAND, Chicago, Ill. (Age 36) (Claims RCA 3.1 RCM 7.0) April 1936 to date with WPA as Dist. Materials Engr., Area Supervisor of Operations, Kane County, and (since April 1938) Field Engr., responsible for field operations for all buildings and structures in Dist. No. 2; previously with American Steel and Wire Co., on valuation of plant and machinery of DeKalb, Ill., plant.

WOULFE FLANAGAN, JOHN HENRY (Assoc. M.), London, England. (Age 57) (Claims RCA 5.9 RCM 16.2) 1938 to date Civ. Engr., and Res. Engr., H. M. Office of Work, secret work in connection with re-armament; previously Cons. Engr., on preliminaries of small water-supply schemes; reconstruction of Islington (London) Metropolitan Borough Council refuse-disposal depot.

APPLYING FOR ASSOCIATE MEMBER

ANDERSON, WENDELL DRUMMOND, Griffin, Ga. (Age 42) (Claims RCA 10.3 RCM 0.0) Sept. 1937 to date Eng. Representative, Southern Vitrified Pipe Association, having full responsibility in engineering field; previously Senior and Constr. Foreman, National Park Service; Inspector of bridges, Iowa State Highway Comm.

BATES, WILLIAM HOWARD (Junior), Statesboro, Ga. (Age 31) (Claims RC 3.5) Jan. to July 1936 Asst. Dist. Supervisor, and Feb. 1937 to date Dist. Supervisor, U. S. Public Health Service, on promotion and supervision of sanitary engineering and malaria control work; in the interim 2d Lieut., 452d Engrs., U. S. Army, Dist. A CCC, Fort Bragg, N.C.; previously Inspector, Georgia State Highway Dept., Atlanta, Ga., on construction supervision and location.

BOUBNOFF, NICHOLAS, Lima, Peru. (Age 41) (Claims RCA 3.2 RCM 0.0) March 1939 to date in charge of design of Arequipa Hospital, Caja Nacional de Seguro Social of Peru; Nov. 1929 to Nov. 1930 Designing Engr. and Nov. 1937 to March 1939 Chf. Engr. with Fred. T. Ley & Co.

BOULDEN, CECIL FOARD, Austin, Tex. (Age 39) (Claims RCA 15.2 RCM 0.0) June 1939 to date Civ. Engr., R. R. Comm. of Texas, representing Comm. as Engr. in dealings with all